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SAN FRANCISCO, JANUARY, 1907

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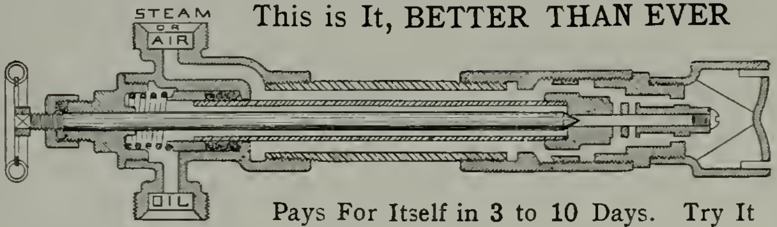

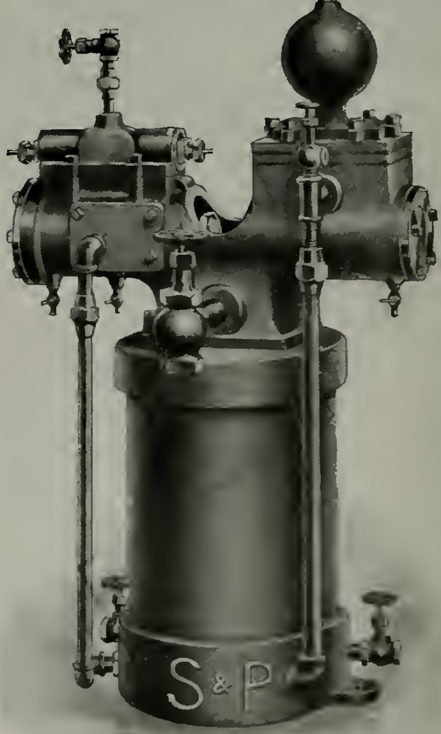

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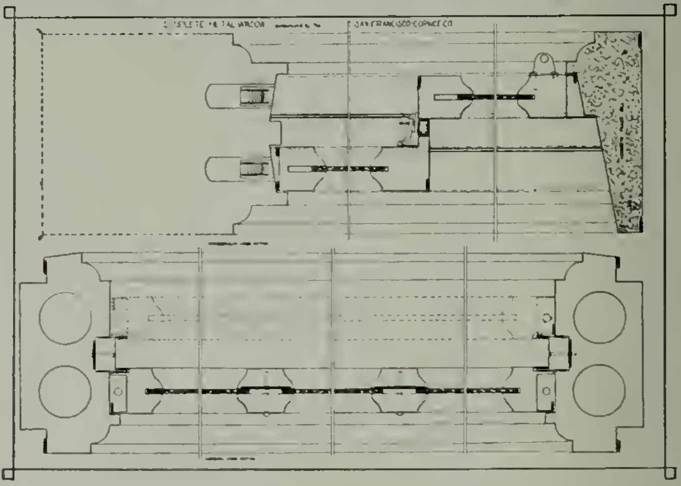

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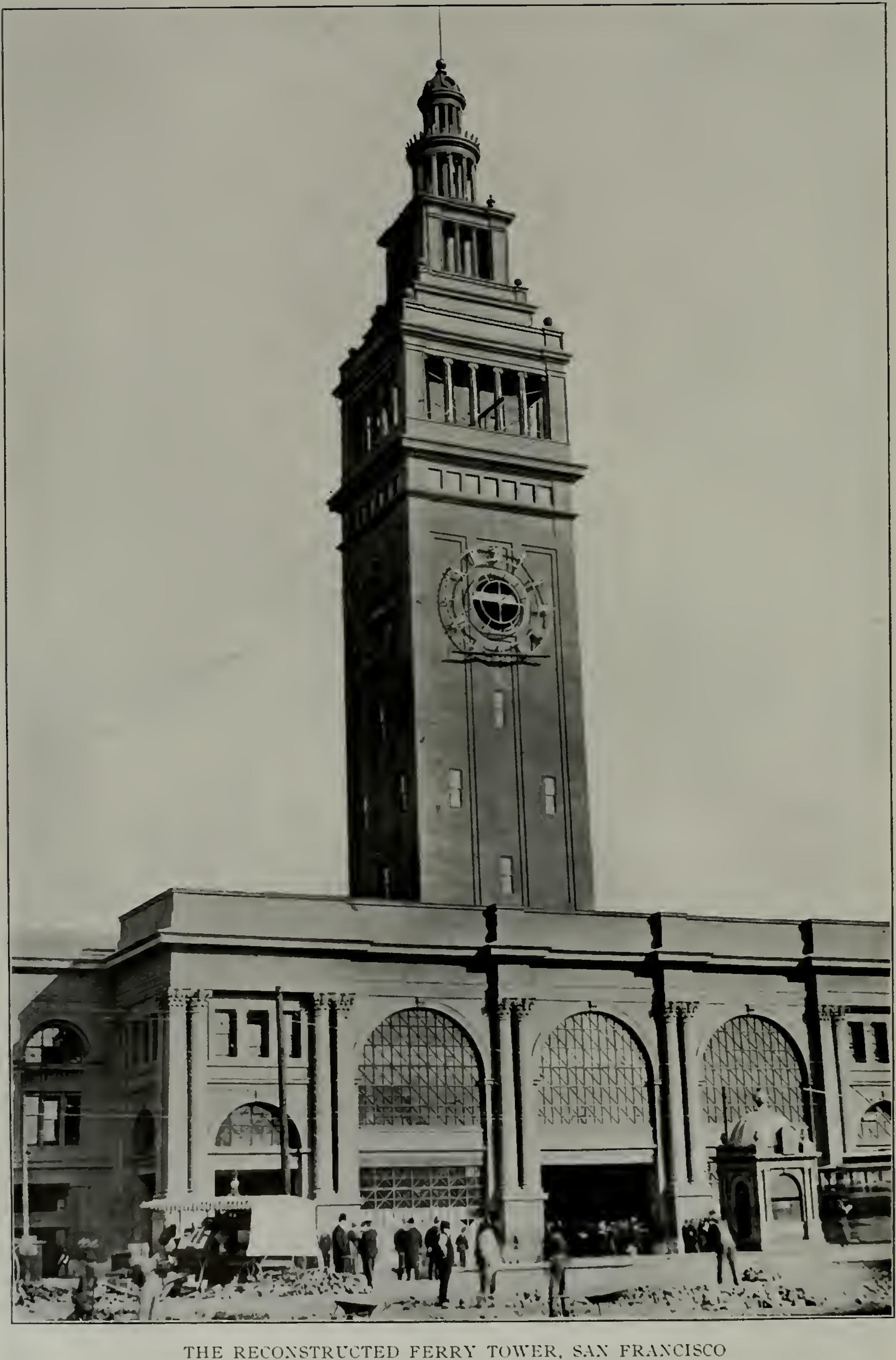

THE RECONSTRUCTED FERRY TOWER, SAN FRANCISCO
December 10, 1906

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AMERICAN BUILDERS REVIEW

VOLUME VI JANUARY, 1907 NUMBER 1

The Tower of the Union Ferry Depot, San Francisco

By F. A. Koetitz

AS is well known, the Ferry Tower received a severe shaking by the earthquake of April last; since then more or less has been written about it and much has been said which does not agree with the actual facts. The writer, having had special opportunity to learn the actual conditions, feels justified in making a statement of them especially as he is a member and the chief engineer of the company which originally built the greater part of the depot building, including this tower, and because the same company has just completed the tower's reconstruction.

Describing the tower briefly, it may be stated that it was originally built with a steel frame enclosed, up to the cornice, by brick walls with a sandstone facing, the walls for each story being practically self-supporting. The frame above the main cornice was covered with galvanized iron, painted to match the stone, as indicated on plan No. 1. The frame was constructed of steel columns, girders, beams and diagonal rods, as shown on plan No. 2, to carry, in addition to the dead load, a reasonable live load and to withstand wind strains of about 30 pounds per sq. ft. of exposed surface. The irregular distribution of the diagonals of the lower stories was made to suit various openings in the walls. The foundation of the tower, as well as that of the building, consists of concrete piers on piles, described in detail heretofore. Although the tower was built several years ago, one of the first thoughts of the writer, after experiencing that never-to-be-forgotten shake was: What must have become of the Depot tower? and he was surprised to find, later, that it was still standing and apparently in fair condition. Immediately after the earthquake the military authorities inspected the Tower, declared it unsafe and ordered ropes stretched about the vicinity to keep the public from approaching within the danger zone in case the structure should fall from later shocks.

The Pacific Construction Company, of which the writer is chief engineer, being most familiar with the structure, was employed, within a few days after the shock by the Board of State Harbor Commissioners to investigate the exact damage and to immediately make sufficient temporary repairs to insure it against falling. Investigation showed that the stone and brick work of the fourth and fifth stories and of a part of the sixth story in the East and West walls were partly thrown down and that much more of the material in these stories was in immediate danger of following. The North and South walls were in better condition, being injured mostly in the fourth story. It was decided to encase these injured stories with boards wrapped with wire cables, to prevent further damage from falling walls and to make the structure safe for further inspection. After this was done, it was found that the main damage was between the third and seventh floors and greatest between the fifth and sixth floors and that it consisted of the following detailed injuries:

All the diagonal rods in these stories were either buckled or injured in some way; one had the eye at one end broken off, in another the rivets connecting the gusset plates and angles sheared off; the stress finding, in each case, the weakest point.

The strain exceeded the equivalent of a 30 pound wind pressure many times as was shown by the elongation of these rods. In five cases, including three 2″ sq. rods, it was necessary to cut off the screw ends to take up the slack. The elongation, in some cases, amounted to 3.125 inches.

On some gusset plates seven $\frac{3}{4}''$ rivets were sheared off. The writer tried to locate the direction of the greatest motion and found that the tower, from the fourth to the eleventh floors was leaning $4\frac{1}{8}''$ southward and 2″ westward, but he was forced to conclude that there was little difference in direction. The rods were

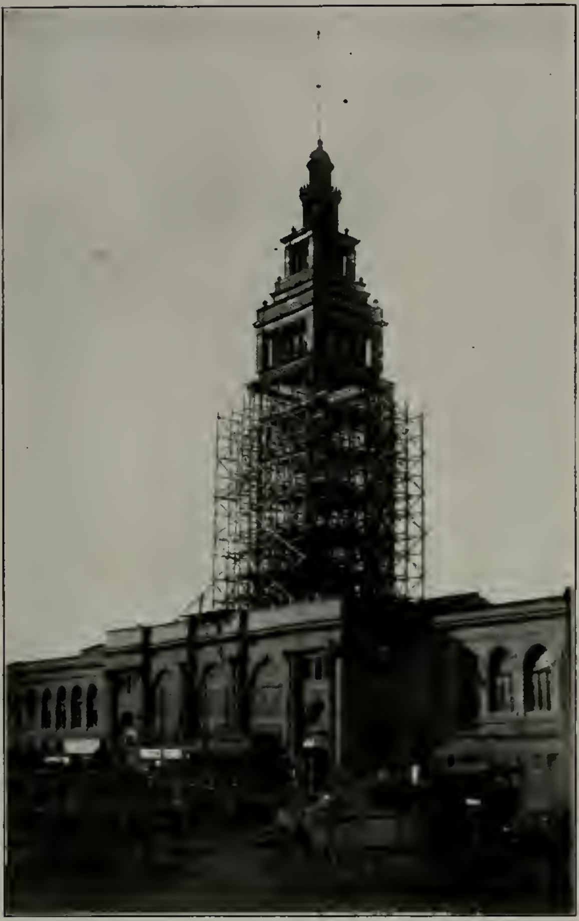


Figure 1.
PROGRESS VIEW
Reconstruction of Ferry Tower, San Francisco

equally slack in both directions in all the four panels of the same story and it is difficult to imagine a movement that would produce such a result without allowing a considerable spreading between columns on the line of the struts or girders.

Five of the girders had the connecting bolts at the columns sheared off and had moved laterally away from the column as much as 2″ At such ends these girders were resting just just one inch on a 3″ angle bracket and one felt like walking lightly on the floors for fear that they might give away and drop floors, girders, walls and men to destruction.

These conditions show that the published reports, saying that the tower was good for many more shakes like the one of that memorable morning, were false, and the writer felt inclined, when these reports were printed, to refute them, but did not do so at that time because he was chiefly interested to complete repairs which would make the tower safe.

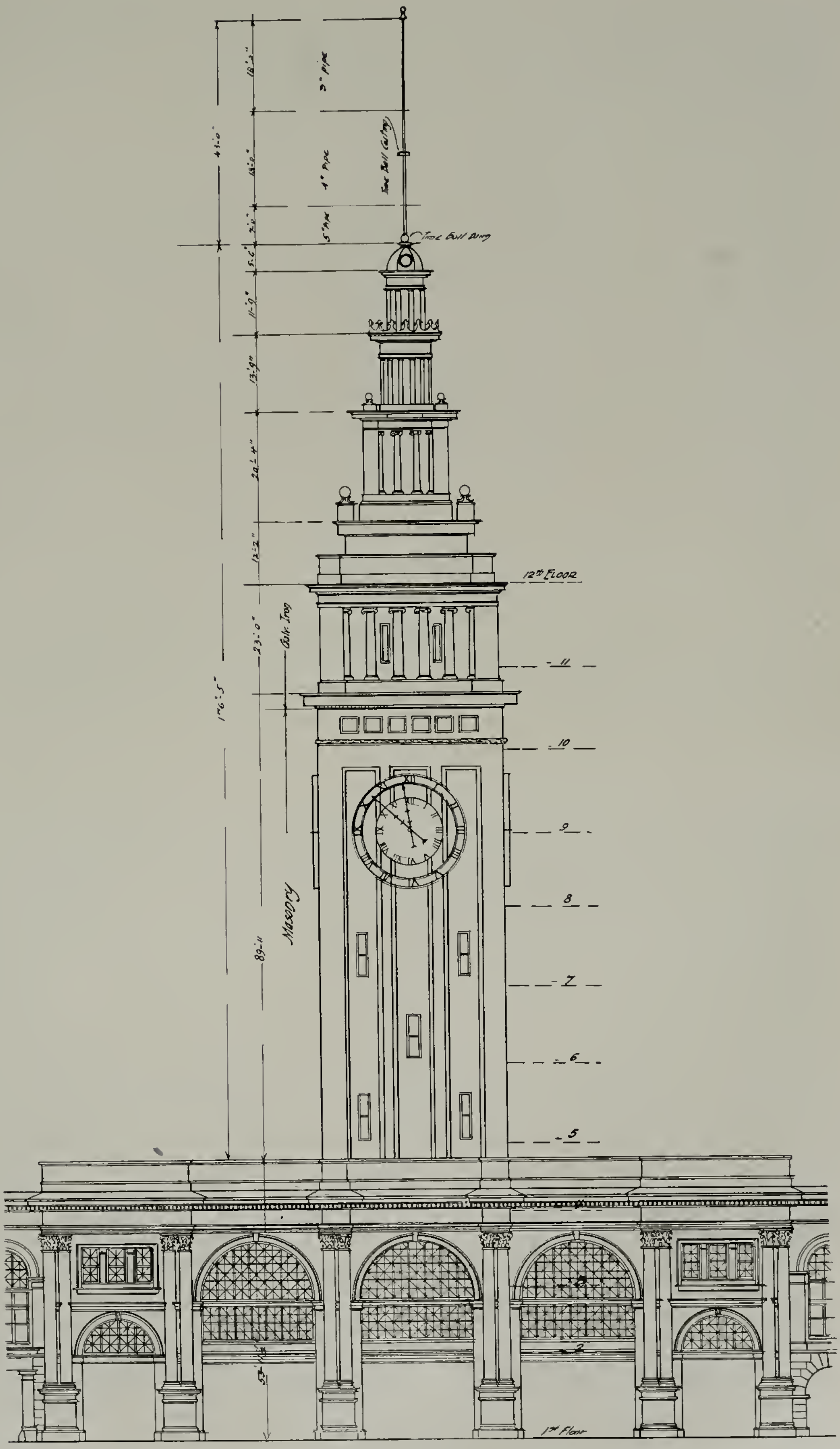
Under the above conditions, it was necessary to provide temporary bracing for the steel frame until the broken walls could be removed and repairs made; wire cables were used, two being attached in place of each rod, and four to each girder that had parted from its column. These cables were twisted tight until they took a good strain. About 9000 feet of cable was used for this purpose and in protecting the walls.

All of the steel frame as well as the stone work above the seventh floor was practically without injury, the greatest damage being the broken eye of one of the rods. This made it evident that the upper portion of the structure was much less affected than the section noted heretofore, but the flagpole gave notice that some movement had taken place up there by developing sharp bends in two directions and by leaning at a pronounced angle. This pole was 42 feet long and was made of extra heavy steel pipe of 5", 4" and 3" inside diameter, the main bend coming near the bottom of the four inch section; this, however, was mainly caused by the weight of a large time-ball head casting which was attached to the pole near the top of the 4″ pipe. It is evident that earthquake shock does not affect the tops of high structures in the same manner as the lower parts; this was proven about a week after the first main shock when the work of rescuing the tower was in progress; a rather severe shock came when the writer and several others were near the bottom of the tower and everyone rushed out of the building, being fearful that the tower would fall, but the men working near the twelfth floor did not know that anything had happened.

Although, as stated above, the upper portion of the tower was intact, it was decided to reduce its weight by taking down all the stone and brickwork, then to repair the steel frame, and to replace the covering with materials of lighter weight. The steel work was carefully repaired; all broken parts were removed and renewed; the separated columns and girders were riveted back to place; all injured rods were repaired, and the tower was brought to proper plumb ready for the covering.

It was, at first, decided to use galvanized iron covering on wooden sheathing, but the writer proposed that reinforced concrete be used, arguing with the Board that while, of course, concrete would be heavier than the galvanized iron construction, it would only weigh about one-half as much as did the former brick and stone, and that it could be made to materially

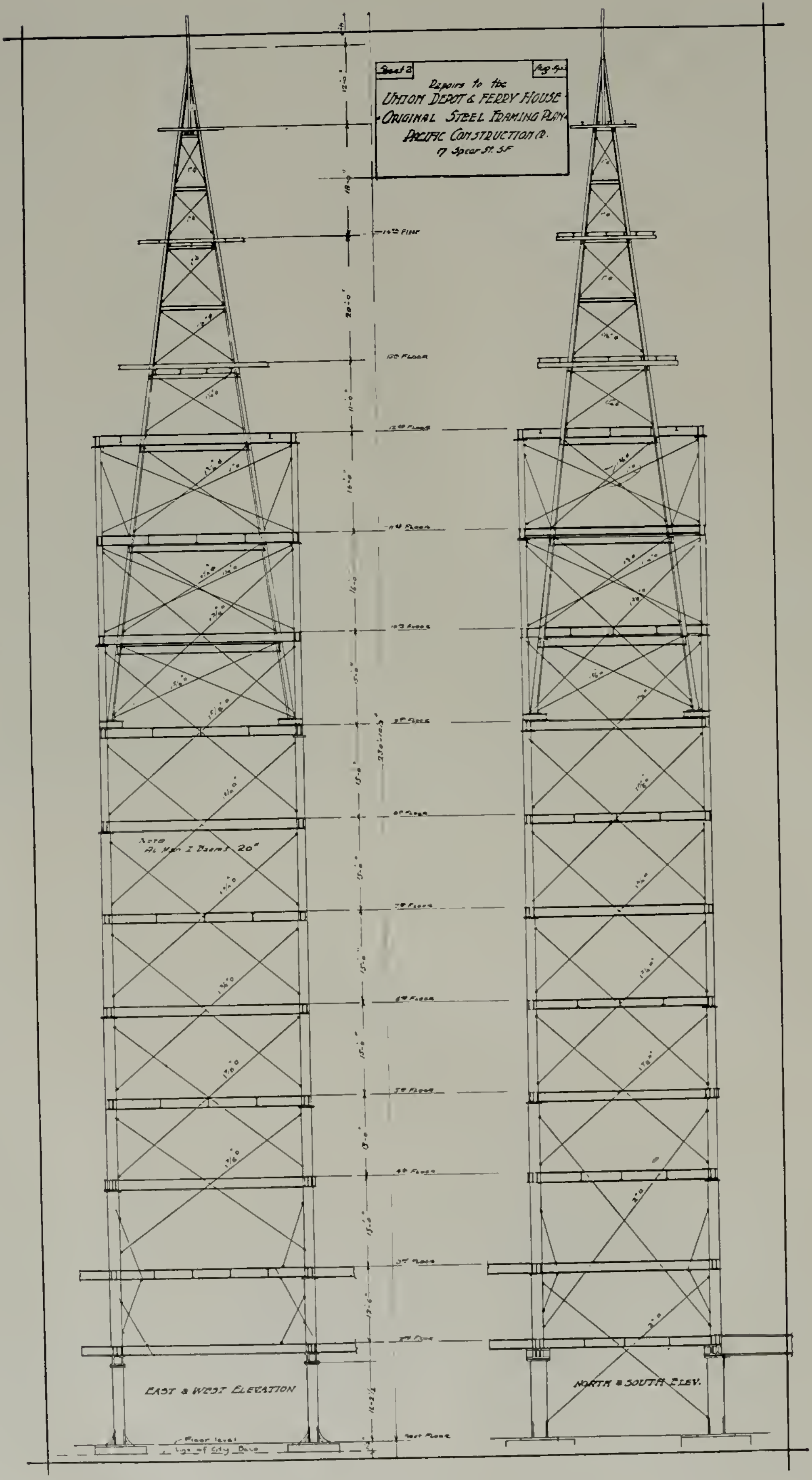
Continued on Page 11



Plan No. 1

ELEVATION OF REPAIRED TOWER

Ferry Building, San Francisco



Plan 2

ORIGINAL STEEL FRAMING PLANS

Ferry Tower, San Francisco

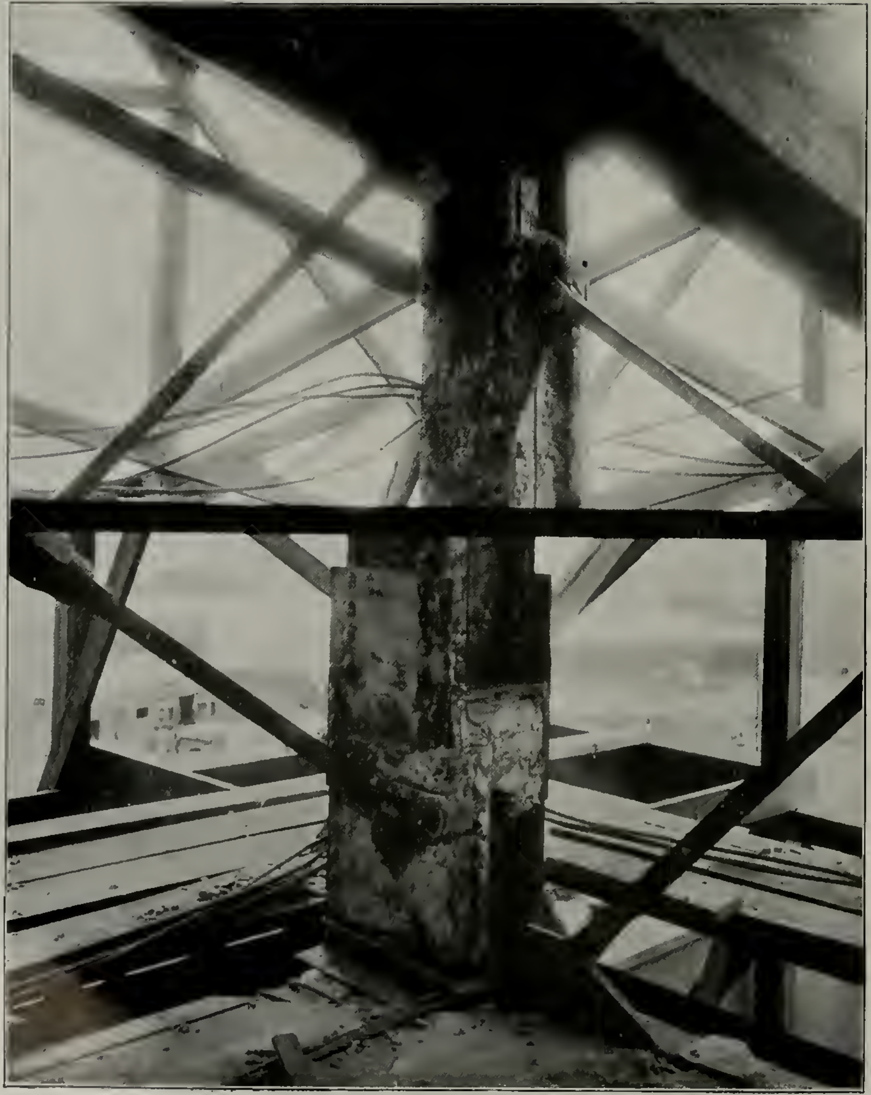


Figure 2

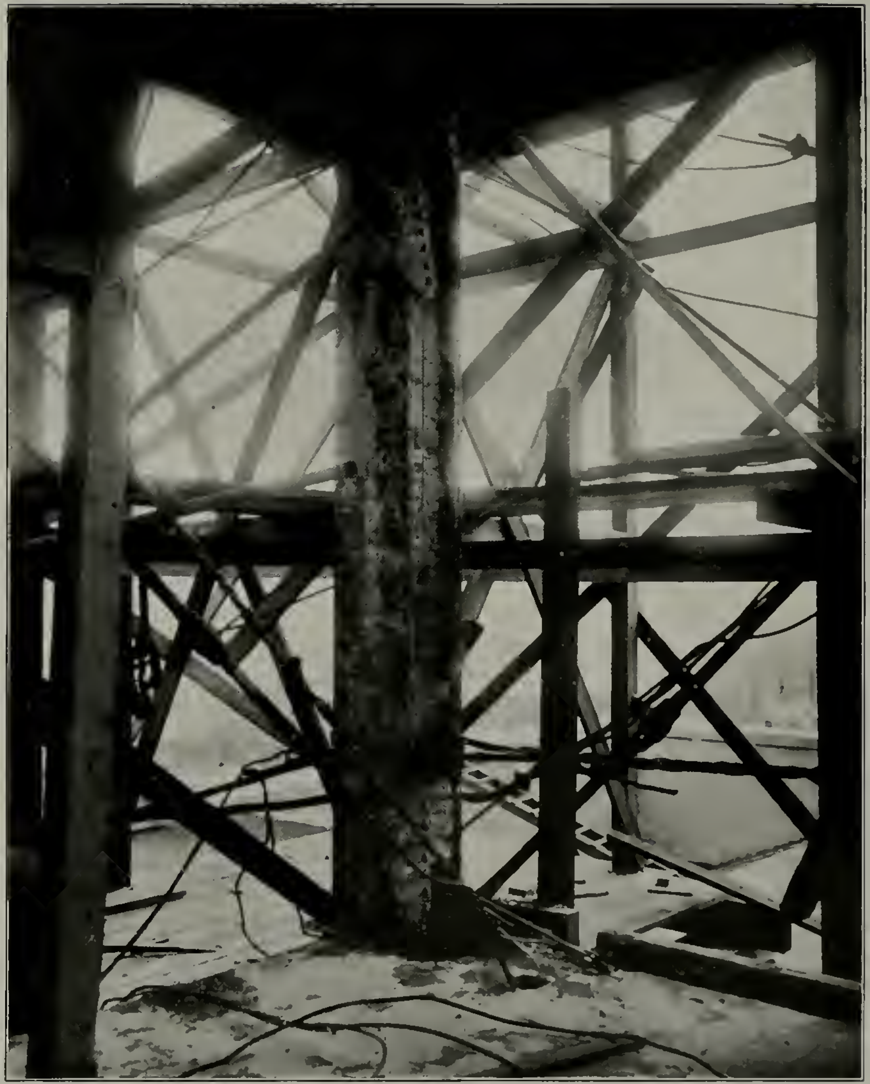
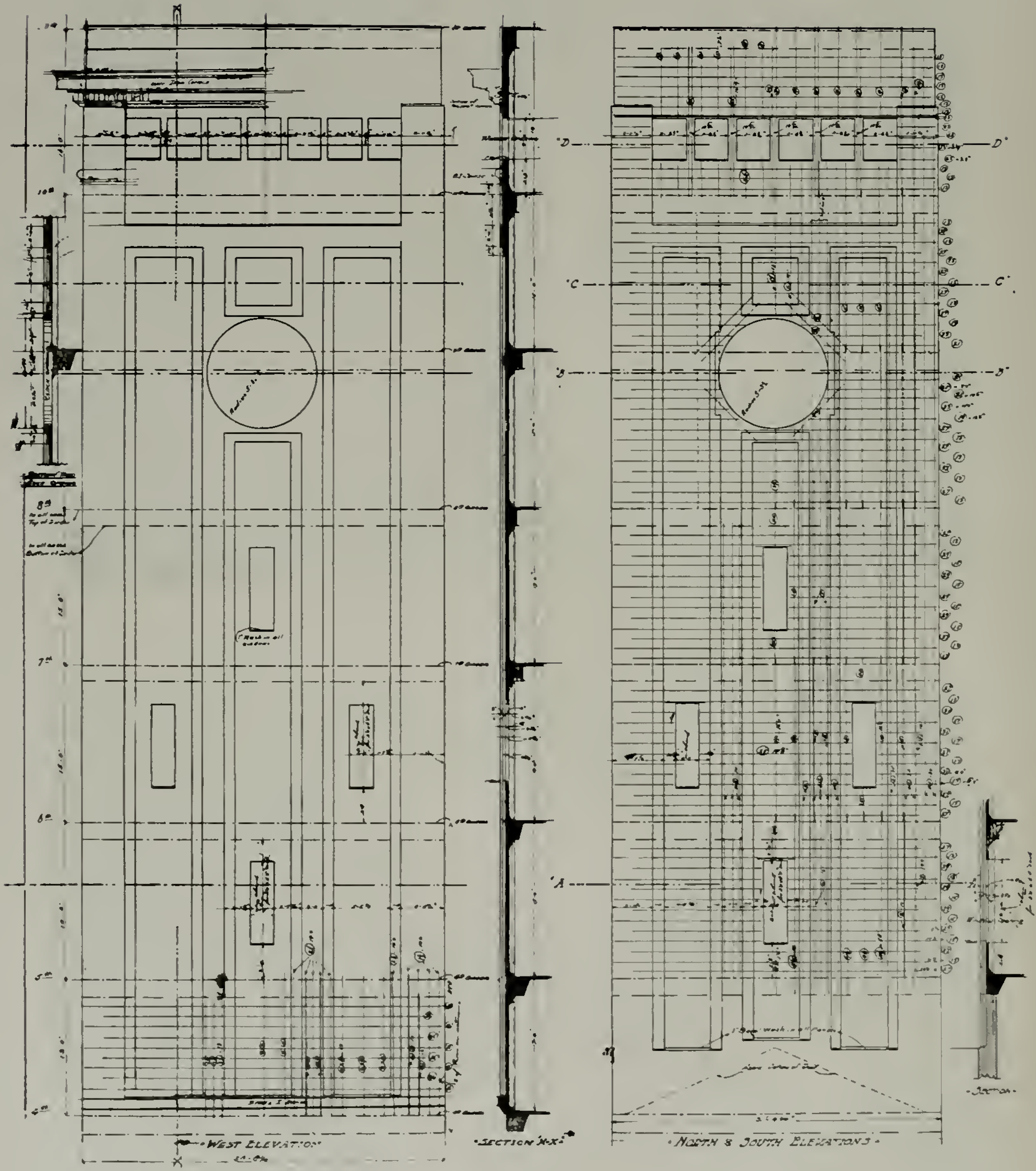


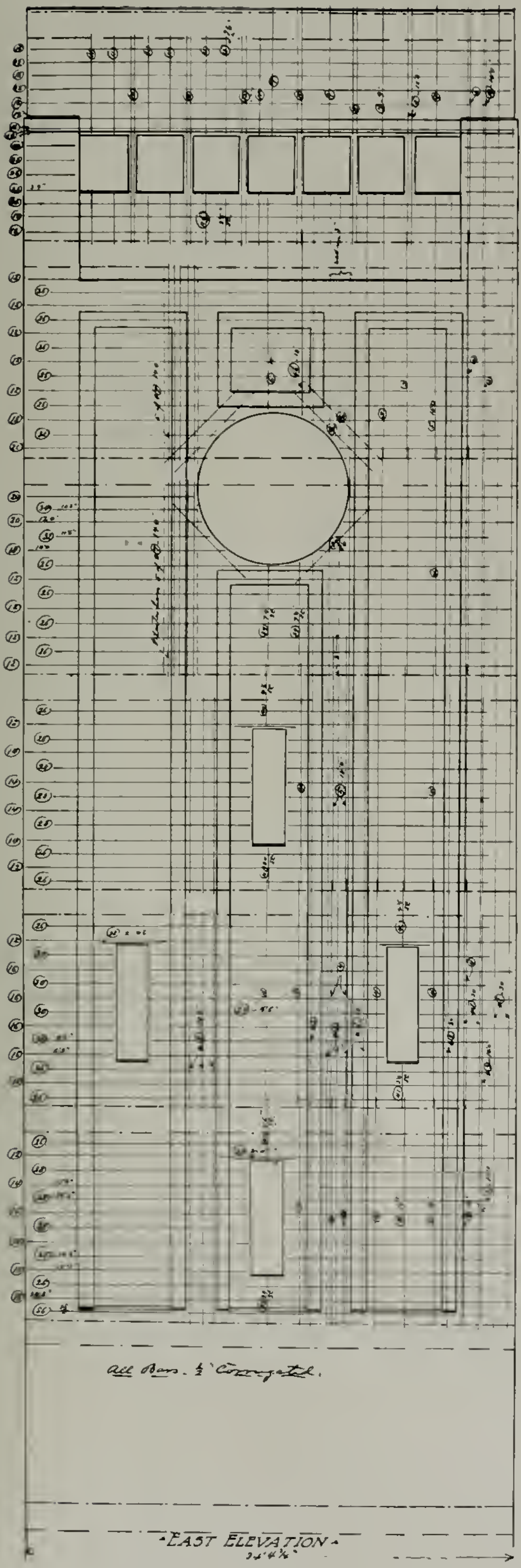
Figure 3

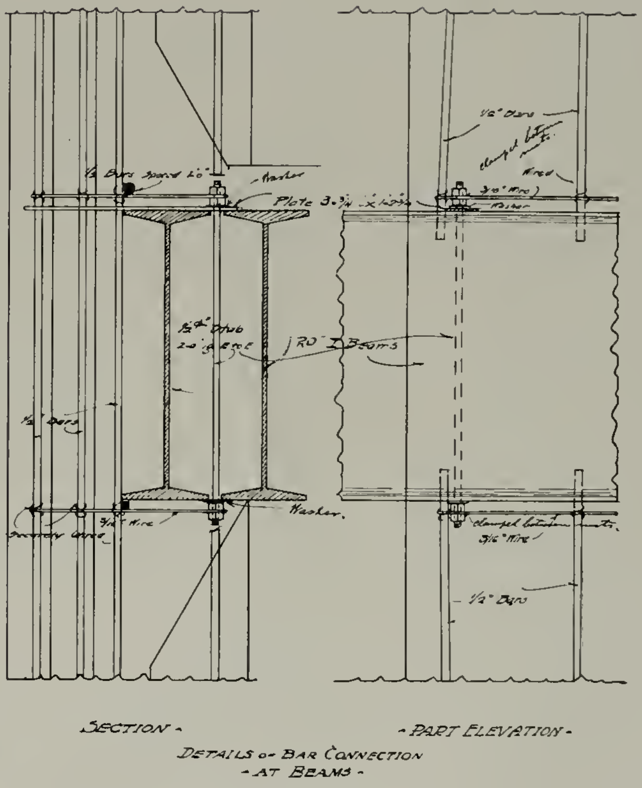


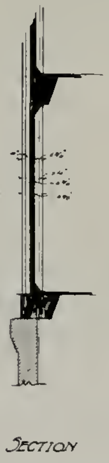
Plan No. 3A

GENERAL PLANS, CONCRETE REINFORCEMENT

Ferry Tower, San Francisco



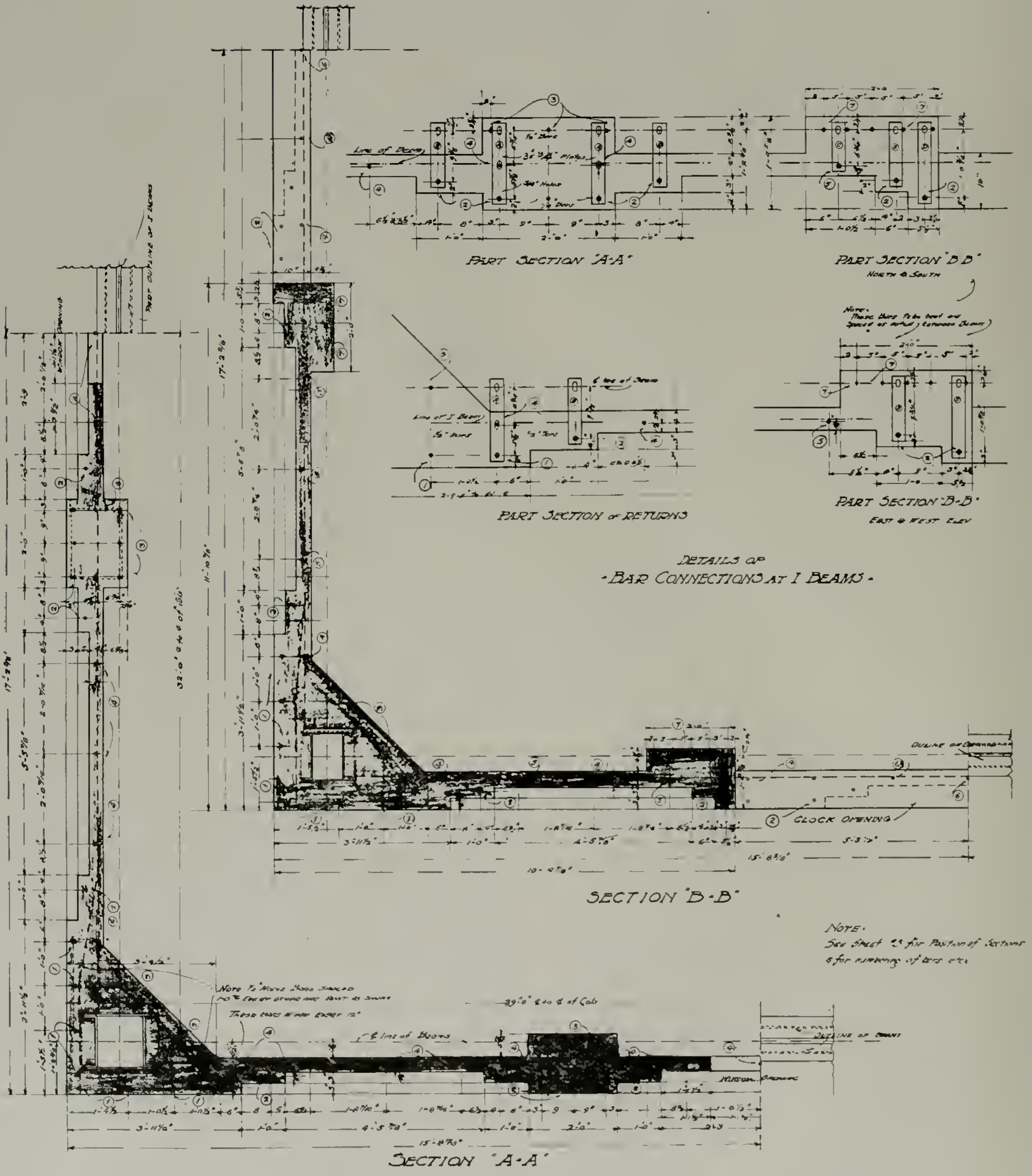




Plan No. 3B

GENERAL PLANS, CONCRETE REINFORCEMENT

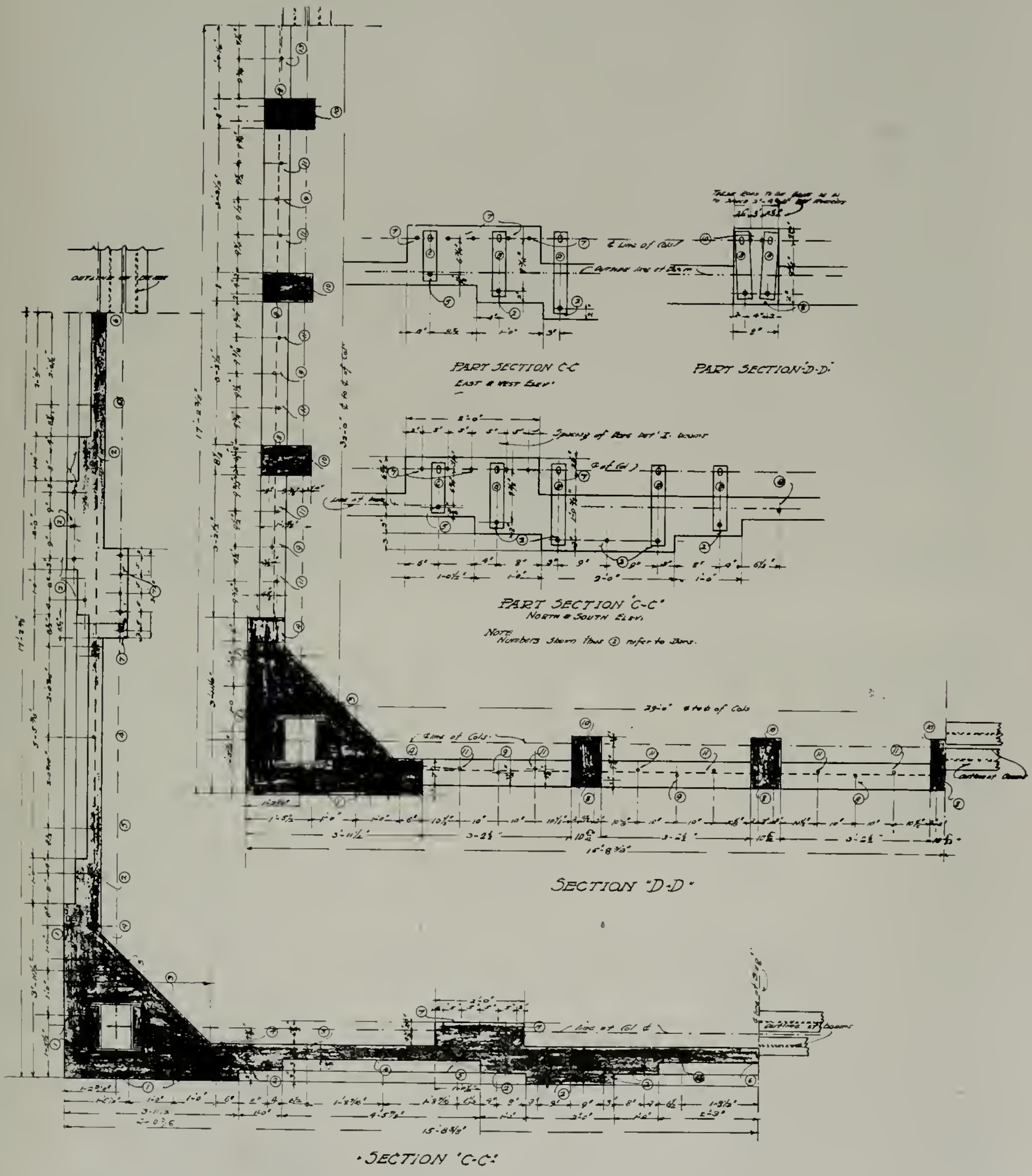
Ferry Tower, San Francisco



Plan No. 4A

DETAILS OF CONCRETE REINFORCEMENT

Ferry Tower San Francisco



Plan No. 4B

DETAILS OF CONCRETE REINFORCEMENT

Ferry Tower, San Francisco

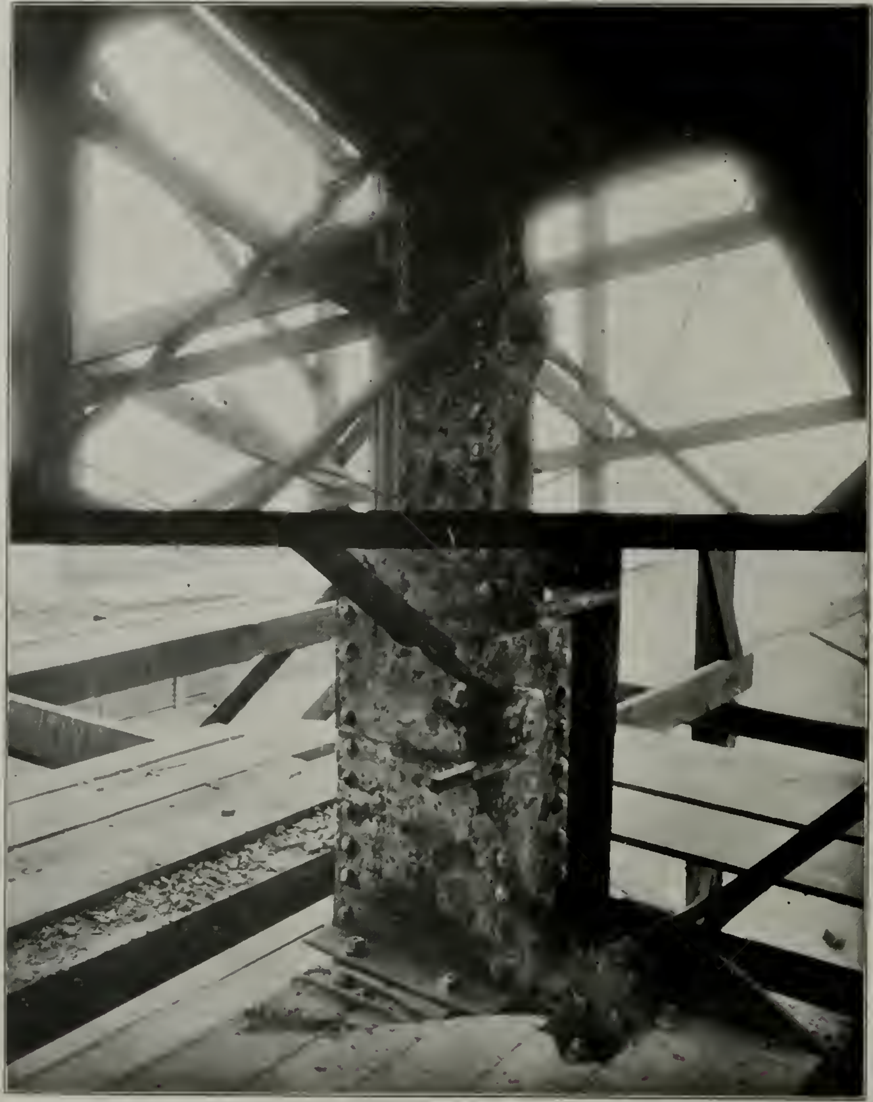


Figure 4

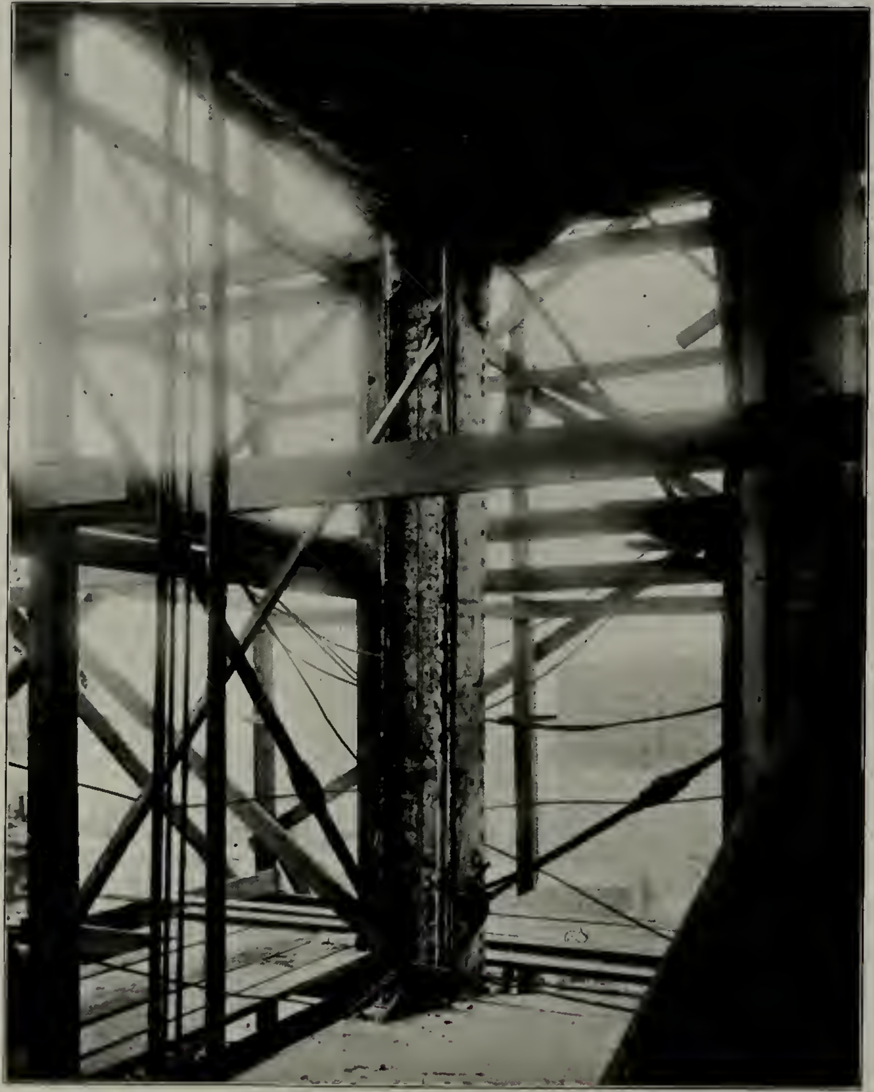


Figure 5

help to brace and stiffen the whole structure. He further argued that such a covering would be practically fire-proof, and could be readily designed to preserve the original outlines of the tower in keeping with the stone front of the main building. He also convinced the authorities that no great reliance should be placed on the capacity of rods which had been strained so much beyond the elastic limit, and pointed out that the reinforced concrete construction would, in itself, almost provide the necessary bracing.

A reinforced concrete design was ordered and its features are shown on plans Nos. 3 and 4. The minimum thickness of panels was made only 4″ in order to reduce the dead weight; pilasters, on the outside, were used to give the tower the same outlines as formerly; pilasters, on the inside, were designed to carry the weight of the walls and to transmit the wind strains to the steel girders. To assist in transmitting strains from the pilasters to the girders, these bars projected outward into the pilasters. The bars were also provided with vertical holes to space the vertical reinforcing rods and to anchor them to the girders. All other rods were securely connected to the steel framework, and the space between the beams forming the girders was filled with concrete to protect all fastenings against rusting. The columns, also, were entirely enclosed in concrete for the same reason after having been first wrapped with expanded metal. In the concrete corrugated steel bars were used as reinforcement and the mixture was high grade German cement, fine washed gravel and sand, with rock crushed to pass through a 1″ ring and over a $\frac{1}{4}$″ screen; one part of cement, three of sand and gravel and three of rock were the proportions, and the concrete was mixed by machine, quite wet, and was deposited into the forms from each floor to the whole story below. The forms were carried up on the outside for the full height, so that the work of concreting could be rushed, when started; the inside forms were built up as the work progressed; this latter form work was difficult and required great care as the diagonal rods of the steel frame interfered in most cases. No attempt was made to finish the outside surface of the concrete, but it was given two coats of Asbestine paint colored to match the sandstone of the main building.

ILLUSTRATIONS.

Figure 1.—Shows the tower with false work around it after it was entirely stripped of the brick and stonework.

Figure No. 2.—At bottom, shows, by the blank holes in connecting angle, where bolts were broken off and girder moved about 2″. At top, to the left, it shows seven rivets of gusset plate sheared off and plate just hanging on one rivet.

Figure 3.—Shows about the same as No. 2

Figure 4.—Shows eyes of 2″ sq. rods, both top and bottom, broken.

Figure 5.—Shows the same as No. 4, top and bottom.

We are indebted to Mr. F. A. Koetitz, Vice-President and Chief Engineer of the Pacific Construction Company, and to Mr. F. M. Butler, Secretary-Treasurer of the same Company, for much kindly assistance in the preparation of this article.

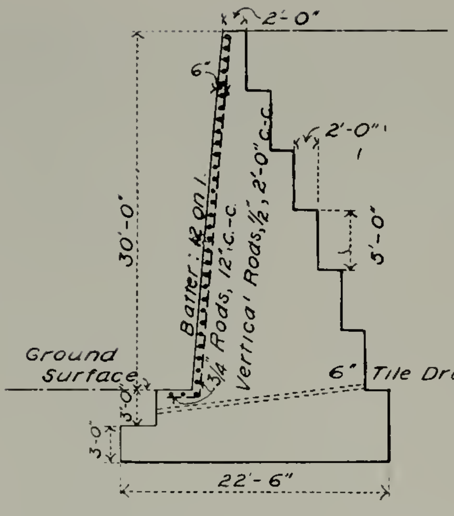
Concrete Versus Reinforced Concrete Retaining Walls

A Study by H. D. Dewell

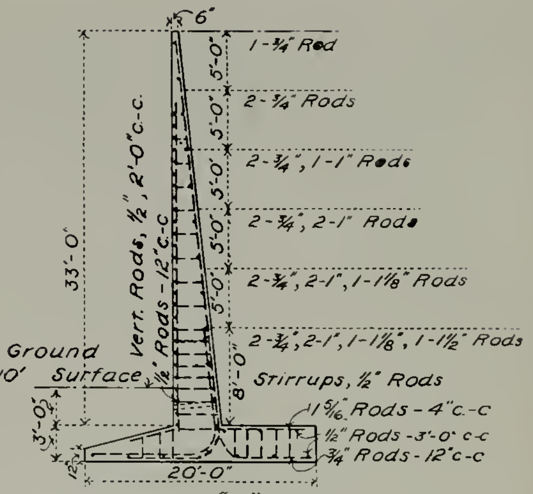
IN the following notes, the writer presents a brief study, made during the winter of 1905–1906, of the comparative costs of three common types of concrete retaining walls; namely, the gravity, the tee and the counterfort wall. The problem may be concisely stated as follows:—

1. Find completely an economic section of a concrete gravity retaining wall, given the ruling dimensions. The retained material is well-drained sandy earth. The head of earth on the wall is thirty feet. The foundation bed is six feet below the finished ground leve at the outer toe and rests upon earth. The resultant pressure on the base must not lie outside of the middle third and the pressure on the foundation earth at the base of the toe must not exceed two tons per square foot. Prescribe and design, if necessary, such longitudinal reinforcement as is required to prevent settling, setting and thermal cracks.

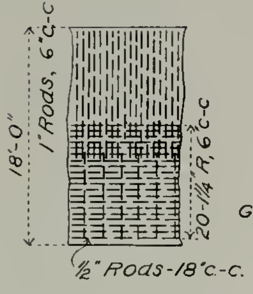
2. Design a tee shaped section, reinforced wall to replace the gravity wall of (1).



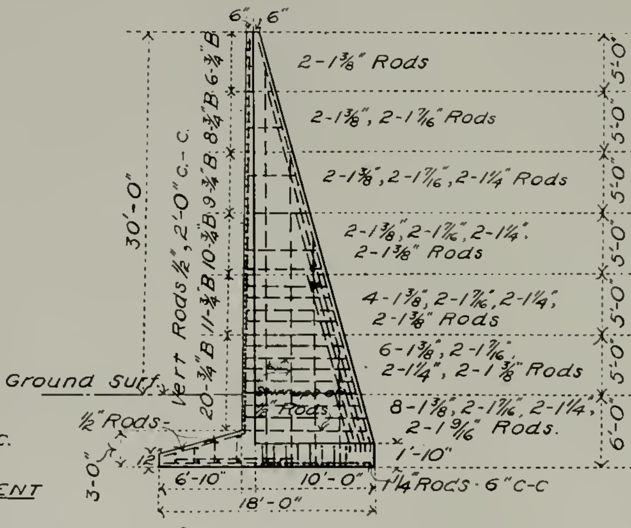
SECTION OF GRAVITY WALL.



SECTION OF "T" WALL.



BOTTOM REINFORCEMENT OF BASE.


6" 6"
2-1⅜" Rods
2-1⅜", 2-1 7/16" Rods
2-1⅜", 2-1 7/16", 2-1¼" Rods
2-1⅜", 2-1 7/16", 2-1¼", 2-1⅜" Rods
4-1⅜", 2-1 7/16", 2-1¼", 2-1⅜" Rods
6-1⅜", 2-1 7/16", 2-1¼", 2-1⅜" Rods
8-1⅜", 2-1 7/16", 2-1¼", 2-1 9/16" Rods.
30'-0"
Vert. Rods ½", 2'-0" c.-c.
20-¾"B, 11-¼"B, 10-¾"B, 9-¾"B, 8-¾"B, 6-¾"B
Ground Surf.
½" Rods
3'-0"
6'-10"
10'-0"
1'-10"
18'-0"
1¼" Rods - 6" c-c
5'-0"
5'-0"
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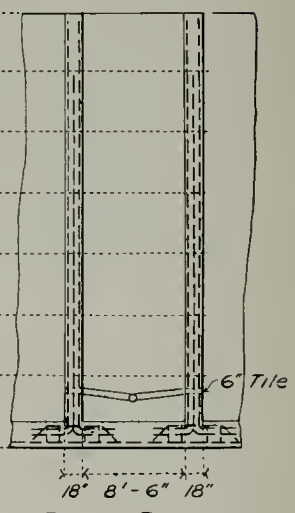
SECTION



REAR ELEVATION

COUNTERFORT WALL.

ALTERNATIVE DESIGNS

Gravity, Tee and Counterfort Retaining Walls

By H. D Dewe

3. Replace the wall of (2) by a similar wall with counterforts.

The value of the earth thrust is computed by the well-known formula of Rankine where:—

$$E = \frac{wh^2}{2} \cos i \, \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}}$$

For the conditions of the problem, the angle "i" has the value of zero, since the surface of the retained earth is horizontal. The angle of repose of the earth is assumed to be 33° 41′, and the weight of the earth is taken at 105 pounds per cubic foot. With these assumptions, the value of the earth thrust is found to be 13600 pounds, and acts in a horizontal direction, at a depth of two-thirds the head of earth, or twenty feet.

The dimensions of the section of gravity wall to resist this earth thrust are shown in the accompanying plate. The resultant of the earth thrust; the weight of the wall, and the weight of the earth resting upon the wall lies inside the middle third of any horizontal section of the wall. For the prevention of thermal setting and settling cracks, steel is inserted in the face of the wall as shown in the drawing. This steel might have been omitted, and expansion joints alone provided to prevent temperature cracks, but the additional cost of the steel is small, and it effectually tends to prevent any unsightly cracks.

In both the tie and counterfort walls the volume of concrete per lineal foot of wall is much reduced, the weight of the earth resting on the wall being utilized to furnish resistance to overturning and sliding. Steel is introduced where necessary to take tensile stresses of bending and shear, and to aid in binding the wall together, that it may resist displacement as a whole. Combining, as before, the weights of the wall, the superimposed earth and the resultant earth pressure, it is found that for the sections shown, the resultant force intersects the base of the wall at a point just inside the middle third, while the maximum pressure at the toe of the wall is approximately two tons per square foot. Thus, both the tee and counterfort walls have the same factors of safety against displacement as units, as has the gravity wall.

It is not intended to give the calculations incident to the design, but it may be observed that the tee-wall consists essentially of a bottom slab, upon which the retained earth rests, and an upright wall proper, which resists the overturning thrust of the earth by means of its action as a cantilever. Both the toe and heel of the base also act as cantilevers, the former tending to be bent upward by the upward reaction of the earth, and the latter to be bent downward by the weight of earth it carries. Hence, steel bars are introduced in the tension side of the beams. The upright wall must be securely tied to the floor slab, and this is accomplished by extending the reinforcing rods of the wall into the slab, and making them act in reinforcing the slab. More steel than is actually required by theory has been used in binding the different sets of reinforcing rods together, and the face of the wall is provided with reinforcement to resist the strains of varying temperature and settlement.

The bending strains which the wall must resist in its separate parts, may be briefly indicated. It is evident that the maximum moment on the upright wall is at the junction of the wall and the slab, and is, in amount, the product of the resultant earth thrust by its distance from such junction, or

$M = 13600 \times 13 \quad 176800$ pound feet

Similarly, the maximum moment on the toe is the product of the average pressure on the toe, the length of the toe, and the distance of the centroid of such pressure from the junction of toe and wall, or

$M \quad 3600 \times 7.5 \times 3.87 \quad 104000$ lb. feet.

The heel is designed on the assumption that it must carry the weight of superimposed earth, neglecting the upward reaction of the earth. The maximum moment, then, is

$M \quad 33 \times 105 \times 8 \times 4 \quad 110800$ pound feet.

Turning now to the counterfort wall, we still have the principle of utilizing the weight of the confined earth to counterbalance the loss of weight resultant from diminishing the concrete section, and in addition, the volume of concrete per linear foot of wall, is still further reduced by replacing the slab action of the tee wall with a slab and girder action. Thus the earth thrust is now resisted by a thinner slab, supported at intervals of ten feet by counterforts, acting as cantilevers. The counterforts also support the floor slab, which is reduced in depth correspondingly. Both the floor and the wall slabs are assumed to be partially fixed at the supports, to the extent that the maximum bending moment is taken as $\frac{1}{10}wl^2$, midway between supports, instead of $\frac{1}{8}wl^2$, as in a simple beam, and rods are introduced over the supports to furnish the continuity necessary to such an assumption. The slab is considered as composed of independent beams, each one foot in width, which is much on the side of safety. In the upright slab, the bending moment on such a series of 12-inch beams varies from approximately zero at the top, to a maximum at the base. This maximum moment is 7200 pound feet.

The maximum moment on the floor slab is

$M = \frac{1}{10} \times 105 \times 34.5 \times 8.5 \times 8.5 = 26{,}200$ pound feet.

The counterfort, acting as a cantilever, must withstand a maximum bending moment of the value

$M = 13{,}600 \times 8.5 \times 14.5 = 1{,}670{,}000$ lb. feet.

Possible conditions of failure by shearing or diagonal tension; rupture between counterfort and wall; counterfort and floor slab; and anchorage of rods are provided for, but such computations need not be discussed here.

In the design, the allowable compression in the concrete is taken at 600 pounds per square inch, and the allowable tension in the steel at 12,000 pounds per square inch. All tensile resistance of the concrete in bending is neglected; the straight line relation between stress and strain is assumed and the ratio of the moduli of elasticity of steel and concrete is taken as 15.

Following is an estimate of the cost of each type of wall per 10-foot section:

ESTIMATE OF COST.

I. GRAVITY WALL.

33—	$\frac{3}{4}$″ Rods	10′	330	1.50	495
5—	$\frac{1}{2}$″ "	33′	165	.67	110

Total Steel 605 lbs.
Total Concrete, 138.3 cu. yds.

II. TEE WALL.

27—	$\frac{1}{2}$″ Rods	10	270	.67	180
5—	$\frac{1}{2}$″ "	28	140	.67	94
1025 Lin. ft.	$\frac{1}{2}$″ Rods	..	1025	.67	687
10—	$\frac{3}{4}$″ Rods	42	420	1.50	530
10—	$\frac{3}{4}$″ "	38	380	1.50	570
10—1″	"	33	330	2.67	880
10—1″	"	28	280	2.67	748
10—$1\frac{1}{8}$″	"	23	230	3.38	776
10—$1\frac{1}{2}$″	"	21	210	6.00	1260
30—$1\frac{5}{16}$″	"	15	450	4.60	2070
22—	$\frac{1}{2}$″ "	10	220	.67	147

Total Steel 8042 lbs.
Total Concrete, 47 cu. yds.

III. COUNTERFORT WALL.

64—	$\frac{3}{4}$″ Rods	10	640	1.50	975
5—	$\frac{1}{2}$″ "	43	215	.67	144
40—1″	"	10.5	420	2.67	1120
20—$1\frac{1}{4}$″	"	10	200	4.17	834
7—	$\frac{1}{2}$″ "	7	49	.67	33
8—$1\frac{1}{4}$″	"	9	72	4.17	300
100 lin. ft.	$\frac{1}{2}$″ "	....	100	.67	.67
2—$1\frac{3}{8}$″	"	38	76	5.05	384
2—$1\frac{7}{16}$″	"	33.5	67	5.52	370
2—$1\frac{1}{4}$″	"	28.5	59	4.17	238
2—$1\frac{3}{8}$″	"	23	46	5.05	232
2—$1\frac{3}{8}$″	"	18	36	5.05	182
2—$1\frac{3}{8}$″	"	12.5	25	5.05	126
2—$1\frac{9}{16}$″	"	8	16	6.5	104

Total Steel, 5109 lbs.
Total Concrete, 32.2 cu. yds.

COMPARATIVE COST.

Gravity Wall, 138.3 cu. yds. at \$10 =	\$1383
605 lbs. Steel at .05 =	30
	\$1413
Tee Wall, 47 cu. yds. at \$10..........	470
8042 lbs. Steel, at .05	404
	\$874
Counterfort Wall.	
Concrete, 32.2 cu. yds. at \$10	\$322
Steel, 5109 lbs. at .05	255
	\$577

The cost of the tee wall is seen to be 61.7% of that of the gravity wall, while the counterfort wall costs 41% of the gravity wall.

Reinforced Concrete Construction in San Francisco

By Horatio F. Stoll

ALREADY it is quite apparent that reinforced concrete is to enter largely in the reconstruction of San Francisco. There is scarcely a block in the downtown burned district that will not soon boast of at least one reinforced concrete building, for they are to be seen on every hand in various stages of construction. The five-story Flannery Building, on the corner of Geary and Market Streets, is the first structure of this kind to be occupied. The Freeborn Estate Company is erecting a three-story office building a few blocks below on Market, and, at the corner of Davis and Market, the Lichstein Realty Company's three-story office building has reached the second story. The Hooker Estate Company is putting up a four-story concrete building on Battery between Pine and Bush, and E. W. Hopkins a seven-story structure on Pine and Leidesdorff. South of Market two pretentious office buildings are rapidly rising. One is the eight-story Bride Investment Company's building, on Mission Street near Third, and the other is the Morton L. Cook building on the west corner of Second and Minna.

The most notable reinforced concrete building which has yet been announced for San Francisco is to be erected on the corner of Fourth and Market streets, the site of the old Flood Building. It will be nine stories high and will cost \$1,000,000. Its exterior, for the first two stories, will be veneered with ceramic tile in rich browns. Above the second story, the entire front will be faced with cream-colored glazed terra cotta in rich detail. The corridors and lobbies will be finished in imported marbles, and six electric high-speed elevators will be installed. All the offices and stores will be trimmed in mahogany. A splendid law library will be installed in the building for the use of the attorneys who may be tenants. The building will have a complete heating , lighting and power plant, sufficient to provide for the needs of all the neighboring buildings.

One remarkable feature of this concrete structure, as pointed out by the architect, Charles F. Whittlesey, is the fact that nine stories are made possible within the limit to which concrete buildings are restricted by the city ordinance—one hundred and two feet. The first story will have a height of twenty feet; the second, twelve feet; and the other stories ten feet each. By an ingenious arrangement of the structure, the fact that the roof is of concrete, makes it possible to dispense entirely with an attic story.

Reinforced concrete experts declare that no other construction will stand fire and earthquake as well as reinforced concrete. And according to investigations made by the California Promotion Committee, it would seem that the facts bear out this assertion. It is well known, for example, that the museum at Stanford University was built seventeen years ago of reinforced concrete, being the first building of its kind in California. As compared with our modern methods, it was a very crude example of reinforced concrete construction. Nevertheless, it stood the earthquake admirably. One statue was thrown from the top and all the marble statuary in the interior was toppled to the floor and broken. The pictures on the walls were swung with their faces toward the wall. However, the building sustained no damage, not even being cracked in the slightest extent. The girls' dormitory was also of concrete construction except in the roof. The roof was badly damaged, but the remainder of the building was but slightly injured.

In San Francisco, the Bekins Warehouse was constructed with brick walls, and reinforced concrete for all other parts, such as floors, girders and interior columns. This building sustained practically no damage, either from fire or earthquake. At the time of the quake, the building was under construction and but two stories had been completed. The first story had already been filled with inflammable merchandise, which was entirely consumed. The building was not damaged in the least.

There were many other buildings in San Francisco having reinforced concrete floors. The National Board of Fire Underwriters report of the San Francisco disaster shows that less than five per cent of these reinforced concrete floors were damaged.

In the Baltimore fire there were two buildings of reinforced concrete in the hottest part of the conflagration. One of these was five stories high, with brick walls, and all the interior construction of reinforced concrete. The brick walls were destroyed by the intense heat, leaving the entire interior construction standing with the full five stories practically undamaged, and requiring only the outer walls to be replaced to fit the building for use. The other building was a bank. The first two stories were of reinforced concrete construction, with three stories above of brick and timber. The upper stories were entirely destroyed, heaping great piles of debris into the top of the two concrete floors. The two concrete stories suffered no damage, not even the woodwork being burned.

In Los Angeles, reinforced concrete has been more extensively used than in any other city in the United States up to the present time. The immense Auditorium Building, which has just been opened with a season of grand opera, is unique in many ways. Being entirely of reinforced concrete construction, it has probably the best acoustics of any large opera house in the world to-day. It is possible to hear the prompter on the stage from the farthest seat in the top balcony, which is 130 feet from the curtain fly. When it is remembered that the limit of range of the ordinary human voice is only seventy-five feet, one begins to comprehend this remarkable fact.

The Auditorium building is further noted for three features which have never been before undertaken in reinforced concrete construction—a concrete roof construction, a great balcony overhang, and cement girders carrying extraordinary loads. The balcony was loaded with a test load of 680 pounds to the square foot and, as the overhang is thirty-one feet, it was expected that the deflection would be considerable. As a matter of fact it was only one-twelfth of an inch on the front. Great trusses were used in this building, 112 feet in span, with a depth in the center of eleven feet. When the false work was removed, they showed so little deflection that it was hardly measurable.

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C. DERLETH, Jr. . . . . Editor

ABNORMAL PRESENT CONDITION OF LABOR IN SAN FRANCISCO

WE are pleased to present the following remarks mainly arranged for us by Mr. M. C. Couchot, an engineer well versed with San Francisco conditions.

The extraordinary conditions which have existed in San Francisco since the fire of April last have brought out very prominently to our attention the laws of supply and demand as applied to labor and materials. Immediately after the fire, when to the praise of San Francisco it may be said that everybody was trying to help his neighbor, labor interests, much to their credit, by the voice of the President of the Building Trades Council, promised not to raise the then existing standard of wages. Unfortunately our experience of the past few months with the scale of wages is not in accord with what might have been expected shortly after our catastrophe. The order of things has changed much since those days and the writer wishes to emphasize the statement that he does not consider labor entirely to blame for the present wage troubles. The cupidity of landlords on the one hand and the imposition of the lumber interests in raising prices on the other, are, perhaps, the main causes for the present abnormal condition in the rate of wages in San Francisco.

As soon as temporary buildings in the burned district began to be erected in numbers, the demand for lumber took on immense proportions, and the lumber community, from the lumberman handling the logs to the retailer who distributes the sticks and planks, was not slow in profiting by the occasion, and lumber prices soared from $16 per thousand for what was called "No. 1 lumber" to $28 and $29 for what was called "No. 2 lumber." The purchaser often found much trouble in getting a good grade of timber. He had to be satisfied with what he could get. It was absolutely necessary that some temporary buildings should be erected. Business had to be re-established at once and at any cost, and parties desiring to build had to submit to the situation, as the contractors were obliged to refuse to bid on a straight contract on account of the unstable condition of prices.

Landlords in the unburned district, finding a brisk demand for their flats, stores, and houses for offices and lodging purposes, naturally were not to be outdone, and they raised their rents accordingly to meet the high prices of necessary repairing and alteration. The writer knows of an instance in which rent was raised 200 per cent. In general, however, he believes the average raise in rents to be from 20 to 25 per cent. The working classes, finding the rents raised, or finding themselves obliged to move to quarters more distant from building operations, naturally retaliated. A raise in wages was a natural consequence.

The first mechanics to ask higher wages were the bricklayers. They did not raise the wages themselves, but had them raised through the bricklayers' contractors. The concrete men, who in most cases represent but ordinary labor, before the fire were paid $3.00 per day. Their wages were raised to $4 in a most arbitrary manner. Finishers receive $6 per day and their helpers $5. It seems entirely out of proportion that a man wheeling brick debris should be paid $2.50 per day, while a man wheeling rock or sand used for concrete work should be paid $4, especially when we observe that the latter is a more pleasant occupation. Plumbers, after a short strike, raised their wages from $5 to $6 per day, and many received more. Plasterers receive $6 to $7 per day, and the writer has paid as high as $10 per day. Structural steel men have been the most modest in their demands, but lately have asked for a raise of fifty cents. Hodcarriers receive from $4 to $5; hoisting engineers $5. Carpenters receive $5 a day, a wage which is not very high for a good workman, but as is well known, so far as the rate of wages is concerned, it makes little difference whether a man has been twenty years at his trade or only a few months. We must pay the same wage whether he is a good or a stupid workman, and therein lies the trouble.

While the writer believes in organized labor and its right to fix wages so that a man may raise his family honestly and decently, he strongly believes that the difference should be made among men. We should recognize the efficient versus the deficient workman. We should make note of the active versus the lazy man. We should appreciate the producer and discourage the laggard. It is in such differentiation that the labor union fails utterly by bringing the good man down to the level of the poor workman.

Another situation which is really a great deal worse than an arbitrary and general raise in wages lies in the observable reduction in output per man since the fire. The men, as a rule (there are of course exceptions), do not seem to care in the least whether they give a just return for the wages which they receive. They seem bent upon getting all they possibly can in money for the least exertion on their part. The saying of labor leaders that a raise in wages does not increase the cost of production because workmen will then do more work is not substantiated by fact, and this observation, in my judgment, applies not merely to one line of trade, but to all.

How long can this condition exist in San Francisco? The writer does not know, but it is certain that these conditions are abnormal, and as such they cannot last. They are merely retarding the best class of construction and are forcing upon San Francisco a poorer and cheaper class of buildings which will be dangerous in the years to come should conditions like those of last April again threaten our city.

The growth in population of cities of similar life and trade, during the past century, has appeared to follow characteristic lines. Such growth may best be shown at a glance by the use of a diagram upon which is plotted a curve with the years as abscissae and the population as ordinates. Of course, all cities do not increase at the same rate, nor do cities increase in size at the same rate at all stages in their growth. Diagrams showing the growths of cities in Europe and America since 1800 may perhaps best be found in books and reports relating to water supply, sewerage and drainage systems. We advise our interested readers to examine some of these books. It will be found that cities of a like character and of similar commercial conditions compare strikingly in their rates of growth at different stages. Such a study will point out the remarkable fact that of all cities of this class San Francisco, during the past twenty years has had an abnormally low rate of growth in population. May we not explain this remarkable situation by the attitude of those interests which abnormally raise wages or influence the supply of materials? These conditions were unfortunately severe before the earthquake, but are worse now. So long as they continue to exist how can we expect Eastern capital to seek our city, and how can we expect to realize that degree of growth in population and increase in wealth to which our city is entitled by its geographic position?

Good Work For Pacific Coast Harbors

THE California Promotion Committee is proving that it is more than an organization for the purpose of advertising California. It is showing conclusively that it is an organization that does things, and not one which indulges in mere resolutions. It is of record that the Committee has never adopted a resolution. It is also of record that while it never endorses projects, it always gets in behind good ones and does all in its power to advocate them.

The most recent work of the Committee that is of importance to the State is the meeting of the Counties Committee at San Diego on the fifteenth of December. The theme of the meeting was "The Harbors of California." At this meeting fifty-one of the fifty-seven counties of the State were represented, and it was one of the most harmonious gatherings that has come together under the auspices of the California Promotion Committee. All the ports of California were represented, and the needs and requirements of all the harbors were fully set forth in papers read and resolutions presented to the meeting.

This being a State organization it was not deemed either wise or expedient to take up any personal schemes or local ideas, consequently all such were first eliminated from consideration by the Resolutions Committee, and then consideration was given to the needs of the harbors as a whole, and it was decided that the first step toward betterment must come from a thorough knowledge in possession of the law-making bodies, and to this end a resolution was framed, and unanimously adopted, calling upon both the United States and the State of California to empower commissions to make thorough examinations of **all** the harbors of the State. It was suggested that the State be called upon to arrange for a set annual expenditure for the betterment of **all** the harbors, and in order that there might be something more than resolutions, the Executive Committee on California Harbors of the California Promotion Committee was instructed to take immediate steps to see that the provisions of the resolutions were carried out.

The convention adjourned but the work has not stopped. Immediately on returning to San Francisco the Executive Committee on California Harbors, composed of R. B. Hale, chairman, James Rolph, Jr., and Douglass Watson, consulted and called a meeting of its Advisory Committee to assemble in Sacramento on January 11th for the purpose of beginning active operations to carry out the desires of the Counties Committee as expressed at San Diego. This Advisory Committee is composed of members representing all sections and all interests in California, and when it assembles it will be one of the most representative bodies ever gotten together in the State.

Following is the personnel of the Advisory Committee:

Marshall Diggs, Sacramento. President Sacramento Valley Development Association.

Edward Grove, San Diego. President San Diego Chamber of Commerce.

E. A. Heron, Oakland. Director Oakland Chamber of Commerce.

Geo. E. Tatterson, Stockton. Director Stockton Chamber of Commerce.

Thomas Magee, San Francisco. Chairman San Francisco Harbor Committee.

J. Downey Harvey, San Francisco. Chairman Committee on Harbors of the Committee of Forty on the Reconstruction of San Francisco.

E. E. Calvin, San Francisco. General Manager Southern Pacific Company.

William Hood, San Francisco. Chief Engineer Southern Pacific Company.

R. P. Schwerin, San Francisco. Vice-President and General Manager Pacific Mail Steamship Company.

Capt. A. H. Payson, San Francisco. Assistant to the President, Atchison, Topeka & Santa Fe Ry Co.

R. H. Swayne, Chairman General Committee on Improvement, San Francisco Water Front.

H. C. Phillips, Los Angeles. Chief Engineer Atchison, Topeka & Santa Fe Railway Company.

W. J. Bartnett, San Francisco. Vice-President Western Pacific Railway Company.

Virgil G. Bogue, San Francisco. Chief Engineer Western Pacific Railway Company.

A. B. Spreckels, San Francisco. Chairman Advisory Committee, the California Promotion Committee.

Edward Mahar, San Pedro.

C. P. Soule, Eureka. Treasurer Humboldt Chamber of Commerce.

Andrew Furuseth, San Francisco. Sailors' Union of the Pacific.

George D. Gray, San Francisco. President Steam Schooner Managers Association.

A. E. Anderson, San Francisco. President Steamboat Owners Association.

W. C. Ralston, San Francisco. President California Miners' Association.

Geo. W. Peltier, Sacramento. President Sacramento Drainage Commission.

D. C. Williams, Santa Barbara. President Santa Barbara Chamber of Commerce.

Howard K. Johnson, Sacramento. President Anti-Debris Association of California.

Capt. Wm. H. Marston, San Franicsco. President Shipowners' Association of the Pacific Coast and the Chamber of Commerce of San Francisco.

L. E. Blochman, Santa Maria. Secretary Santa Maria Chamber of Commerce.

E. W. Clark, San Luis Obispo.

Some Observations on Fire Protection

By E. S. Hand*

TO compass within the bounds of a paper limited to a fraction of an hour, as presentation to this body must be, a comprehensive review of the problems of fire protection is quite impossible.

So much and so forcible attention has been directed to the subject, by reason of the extravagant fire losses experienced in the United States during the last decade, that the matter has ceased to be one wholly of academic interest to the fire underwriter, and has assumed its rightful position as one of the most important functions of the national economy. For, when it is considered that within the space of a few hours one of the chief commonwealths of the country may be completely burned over—physically annihilated by a conflagration in all probability quite preventable—most other public questions pale by comparison.

Any general appreciation that fire loss may be mitigated is of comparatively recent birth and for such knowledge we are indebted largely to the fire underwriter. The modern fire insurance organization, growing, as it did, out of practically "a game of dice at Lloyds," is no longer an accident but an enduring expression of an urgent commercial need. And it was not within the range of possibility that any extensive campaign of indemnity against fire loss should be conducted without a realization that much of such damage is quite preventable, and to the investigation of problems along these lines, bureaus and organizations all over the world are devoted. Most of such bodies are connected, directly or indirectly, with fire insurance companies, and upon their findings together with acceptance of these findings by the insured public, depend the rate demanded for indemnity. All this has been a most salutary course of public instruction. It has presented very clearly the principle that a voluntary decrease by the insured of the fire risk upon his property warrants a reduction of the fire insurance rates on the part of the underwriter—perhaps not corresponding but certainly in the same direction. A necessary corollary of this is a realization that if fire protection be of a given value to the underwriter, it must be of even greater value to the owner, and this is the meat of the whole situation. A dawning realization of this fact finds expression in the Code Napoleon, whereby, in effect, each man was held responsible to his neighbor for fire occurring on his own premises. Considerably later came the underwriting bodies before referred to, whose function was to justify the insurance rate with the hazard undertaken, and to-day we have in this country above 150 fire insurance organizations, practically all affiliated in the study of fire loss reduction, and all encouraging in the largest practical measure, the adoption by the owner of measures of safety.

In addressing this body I may safely assume

EAST, OFFICE TELEPHONE BUILDING
Hyde Street, Near Sutter

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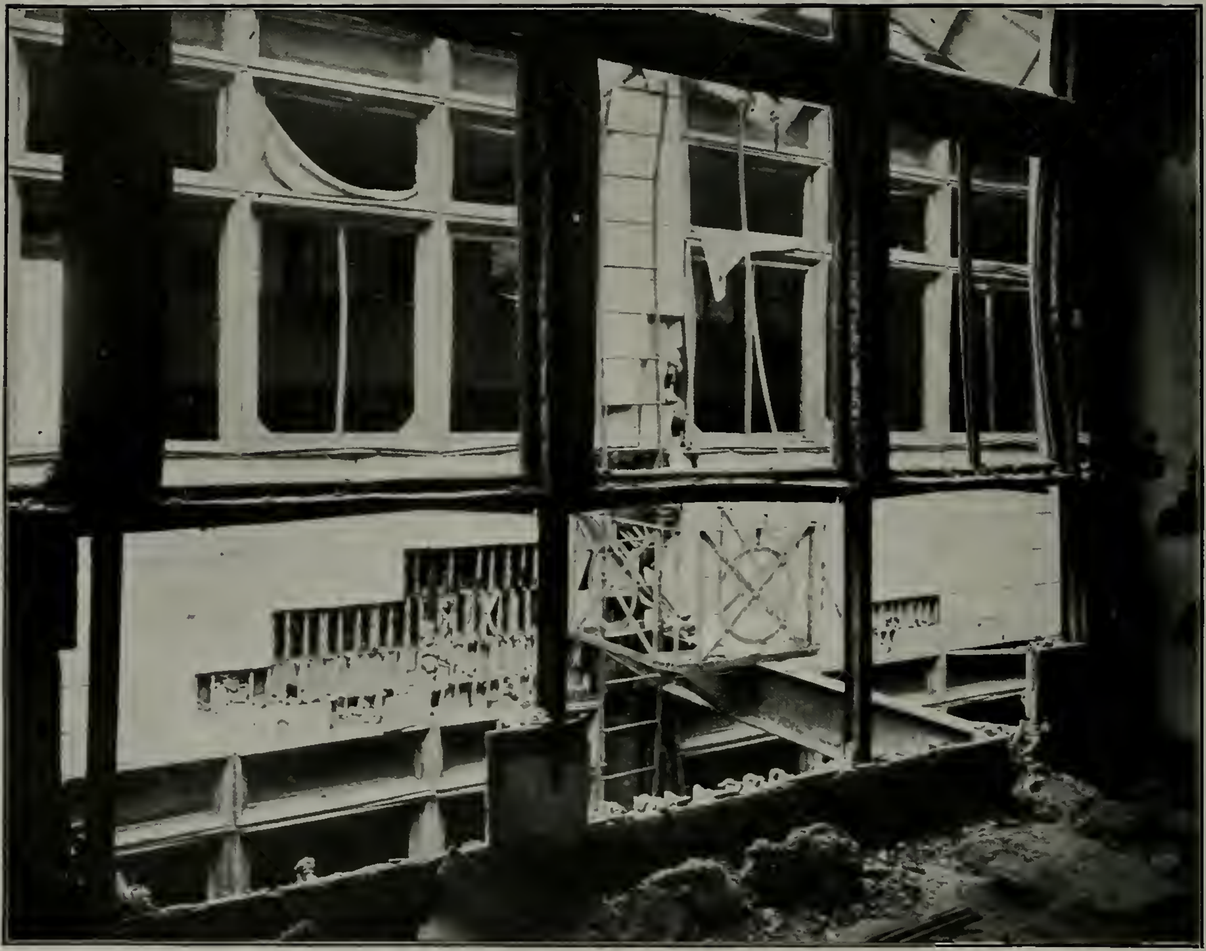
* A paper presented to the Structural Association of San Francisco; Meeting of September 20, 1906.



LIGHT COURT, WELLS-FARGO BUILDING

Wire Glass in Metal Frames

that a general knowledge of the problem of fire loss reduction is possessed, that the value of the science is conceded and that, particularly in view of the experience of the past few months, you are keen in the search of anything which will tend to reduce the likelihood of a repetition of the great conflagrations of the past.

No better statement of the ultimate remedy has ever been made than by the late Edward Atkinson, who summed up the situation as follows: "The only persons who can prevent loss by fire are the owners or occupants of buildings. Upon them rests the responsibility for heavy loss, if any occurs, in nearly every fire. It is the duty of the owner to build of such materials as will discourage the start of fire, and to provide such devices as will prevent the spread of fire. And it is the duty of the occupant to acquaint himself and those under his control with the functions of such materials and devices, to the end that they may be employed promptly and efficiently in the moment of emergency."

The great San Francisco conflagration contributed little to the science of fire protection, as the immediate breakdown of the water distribution system introduced absolutely artificial conditions, and with the preponderance of wooden buildings—over 90 per cent—a free sweep of the fires following the earthquake was a foregone conclusion. As in Baltimore and other great fires, however, examples were evidenced of buildings equipped to withstand the attack of fire, and in their survival, showing clearly what might have been realized had such construction and equipment been universal.

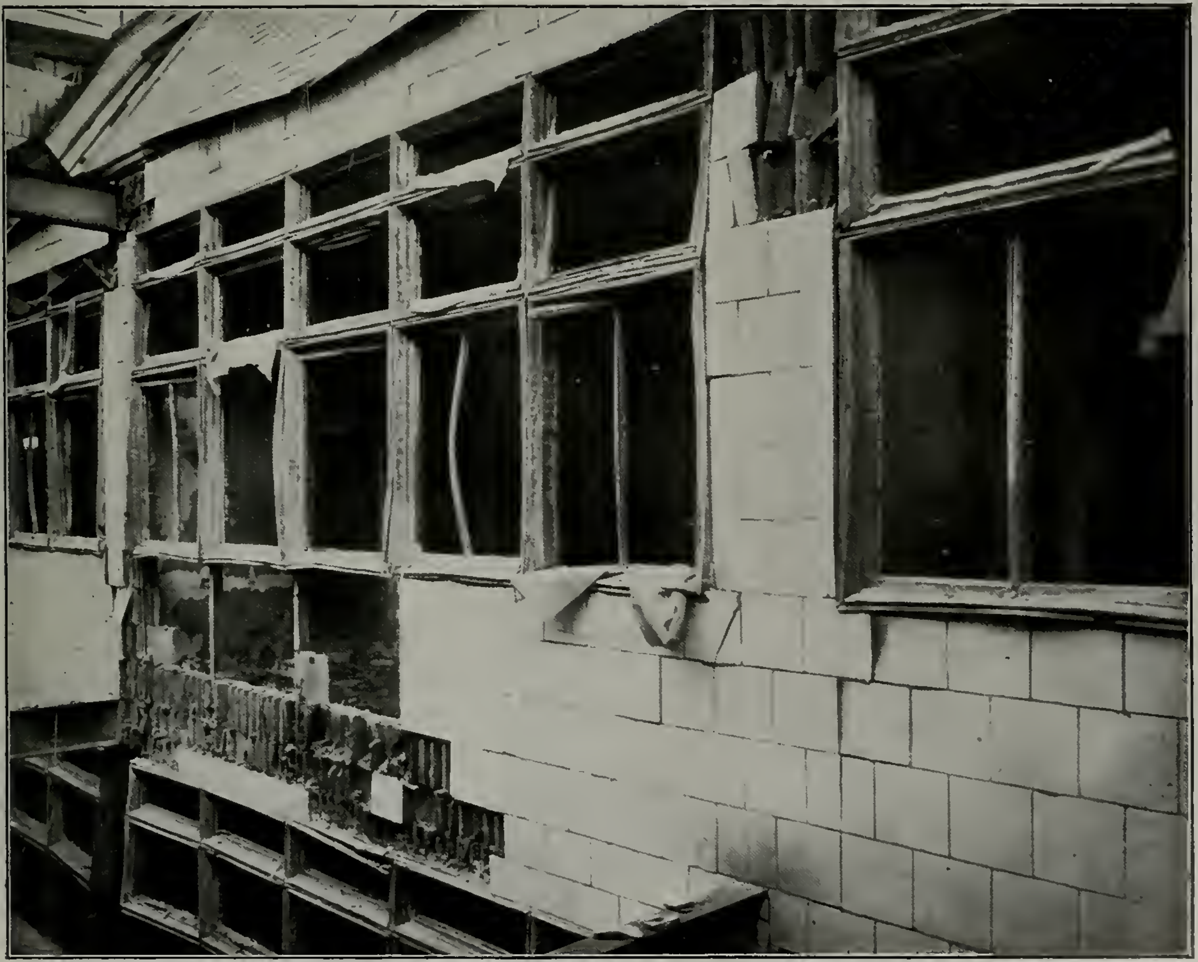
In a general way the science of fire protection has two phases. First, the suppression of fire within the confines of the premises upon which it originates. Second, the prevention of the spread of fire from one structure to another. Within the first phase lie all the problems of fire retardant construction and equipment; in the second are included means and devices which isolate one structure from its neighbors. It is my purpose to touch briefly on this problem of isolation, referring to some instances experienced in San Francisco and other conflagrations.

Statistics covering twenty-five years indicate that about one-third of the total fire loss is due to the contribution of flame from one building entity to another. This tendency toward the spread of fire from building to building is generally termed the exposure hazard and is the real limiting condition which determines great conflagrations.

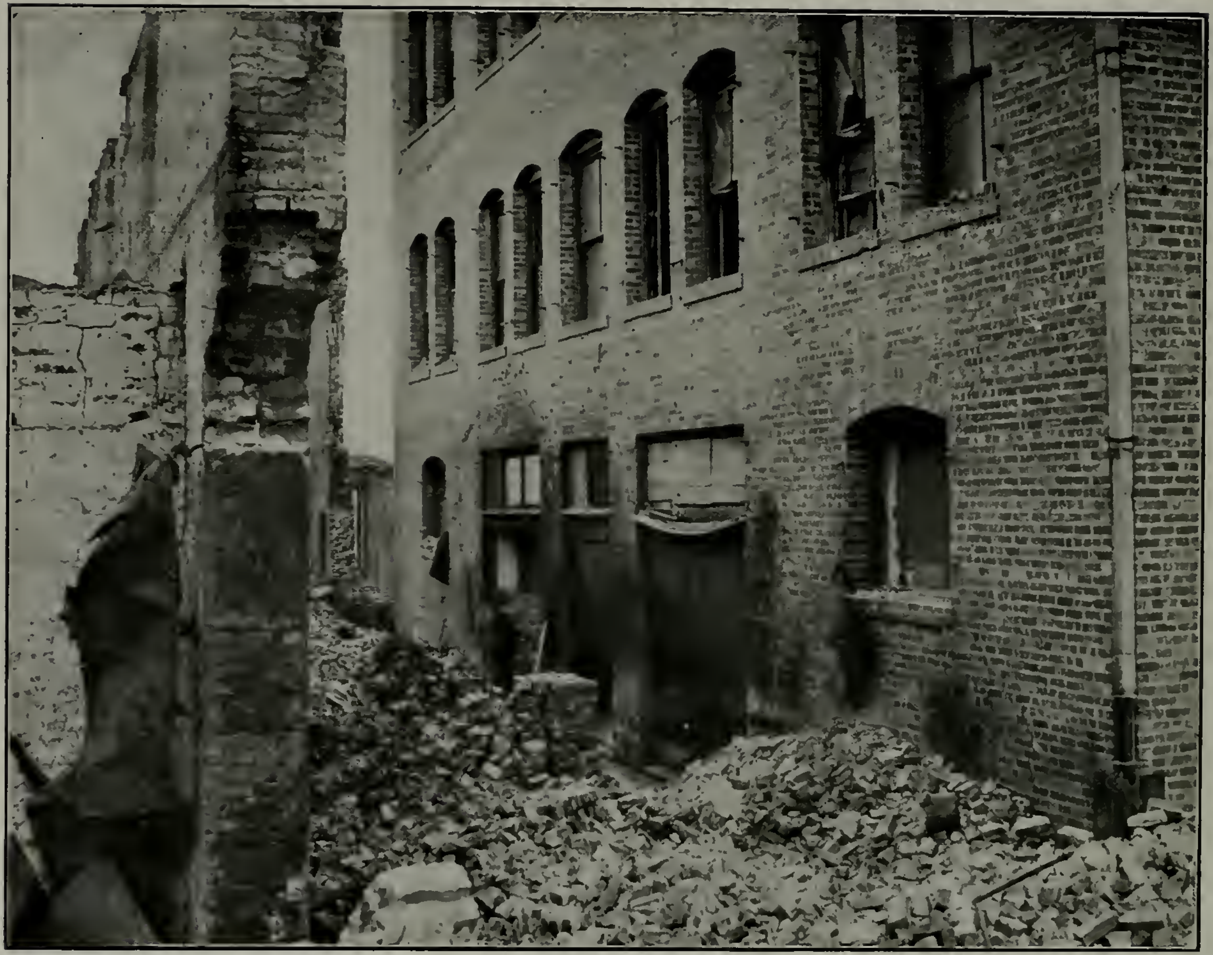
The conflagration hazard is the inevitable accompaniment of the congestion of cities; it is a varying and variable factor and is dependent upon width of streets, efficiency of fire departments and, most of all, on the ability of the individual building to withstand, practically unaided, the assault of fire from without. Given this latter strength, the probability of extensive damage is remote, for the fire cannot spread faster than an efficient department can handle it.

With the continual accumulation of experience afforded by fires, both great and small, specialization in the science of fire protection becomes inevitable. Nothing could be more impressive than the enormous advance in fire resistant construction developed in the United States during the past fifteen years, through the medium of steel, protected by hollow tile or concrete.

With the early accomplishment of fire resistant construction, came the belief that if a building resisted successfully a fire within its own walls, it was immune from conflagration. The first serious jolt administered to this belief came with the experience in the Baltimore conflagration, where many steel skyscrapers, the equal in design and construction of any in the country, not only failed to resist the attack of outside fire, but, when once alight, actually contributed to the spread of the flames. As



WELLS-FARGO BUILDING, LIGHT COURT



VOLKMAN BUILDING, REAR VIEW

Jackson near Sansome Street; Showing Kinnear Doors and Wire Glass Windows

perhaps never before was the lesson borne in at this time of the vital necessity for window and door protection, if buildings are to be in any large degree resistent to the attack of fire from without.

It is manifest that the most efficient armor against the attack of exterior fire is an unbroken brick wall. For obvious reasons, however, buildings for practical habitation cannot be so enclosed, but must be provided with openings for light and ventilation, and for purposes of entrance and egress. Inasmuch as the brunt of exterior fire attack is horizontal, the protection of window openings becomes the most vital feature in reducing the exposure fire hazard. Two types of window protection are in general use. First, the fire resistant shutter, either of metal or of wood covered with metal, arranged to be opened and closed as circumstances demand. Second, a metallic window frame and sash glazed with wire glass.

Of the comparative advantages of these two types of protection a word will suffice. If in position when the attack of fire occurs, and in a good state of mechanical repair, there is comparatively little to choose between shutters and wire glass windows. Certain fundamental conditions, however, operate to the detriment of shutters as a class. First, constructed of metal and hinged, to be opened and closed with more or less frequency, they do not represent an integral part of the building and almost invariably, in practice, suffer from undue corrosion. Requiring to be opened for the admission of light, shutters, unless promptly closed in the moment of emergency, must fail of their protective functions throughout approximately 50 per cent of the time. Statistics indicate that there is not a wide difference between the percentage of day and night fires, and it is evident that unless shutters be closed promptly in emergencies, their usefulness is correspondingly curtailed.

Wire glass windows, on the other hand, are installed in an integral portion of the enclosing wall of the building and receive the same care accorded to the building as a whole. Wherever designed for fire protection purposes, wire glass windows are arranged to close automatically at the approach of heat and their maximum efficiency is thus realized throughout 24 hours of the day.

It may not be amiss to say a word concerning wire glass as a commercial product. While wire glass was not the direct product of a distinct and urgent demand, its adoption for the purpose of fire protection fills a need which is growing with great rapidity. The tendency of modern building construction is toward incombustible elements with very large light openings, accentuating thereby the exposure fire hazard. So far as we can see at the moment wire glass represents the most efficient protection which physical limitations will permit for such openings.

Wire glass, as you undoubtedly know, is plate glass, having a wire netting imbedded between its two surfaces during the process of manufacture. When made properly the two materials become as one, combining the strength of the wire netting with that of the glass. The glass plate is homogeneous and solid and the wire so thoroughly covered as to obviate the possibility of rust or corrosion. Wire glass, when thus made, will crack, but will not scatter, as the separate pieces of glass, in the event of fracture, are each held in position by the wire mesh. When subjected to intense heat, the glass will crack indefinitely, but will re-crystallize instantly when water is turned upon it, recovering its original form of a solid sheet. This property of maintaining the entity of the sheet under extreme conditions of heat, make wire glass one of the most valuable of fire retardents. When suitably mounted in metal frames, it will withstand any degree of heat encountered in an ordinary conflagration, and serves as a protection for window and sky-light openings in a manner closely bordering on the ideal.

If it were possible to bring into being all conditions indicated by the science of fire pro-

CALIFORNIA ELECTRICAL WORKS

Folsom and Second Streets

INTERIOR VIEW, FAIRMONT HOTEL

tection, a practical elimination of extensive fire loss would, no doubt, result. Like many other problems, however, only an approximation at best, can be realized. It is quite out of the question to reconstruct on fire retardant lines, buildings already in existence. It is possible, however, where such structures have brick or stone walls to effectually protect all window openings, thus isolating each from possible fire contributed from its neighbor. Such separating devices may be easily and cheaply installed in all buildings having fire retardant walls and on the advantage of such isolation it is unnecessary to dilate.

You are all familiar with the example of the so-called fireproof buildings, both in this city and in like conflagrations elsewhere, which, after the stress, raised their gaunt shells stripped of everything but the steel and its enclosing clay. On the other hand, Baltimore furnished one and San Francisco at least one other example of a structure not highly fire retardant, in general type of construction, but having good brick walls and all of the outside openings therein sufficiently protected against the assault of fire. In both these instances the structures stood immediately in the path of the conflagration at its severest and by reason of the efficiency of the armoring shell, came unscathed from the test. In San Francisco, the building referred to is that of the California Electrical Works and of its performances the official report to the National Board of Fire Underwriters says:

"The most remarkable instance of successful individual defense was that of the California Electrical Company's factory which manufactured electrical supplies. It was a four-story brick, mill construction plant, with automatic sprinkler equipment, 50,000 gallon tank, elevated 30 feet, 120,000 gallon covered yard reservoir, electric pump supplying tank, yard and private street hydrants. They also had an equipment of private hose. The current for the pump having failed, the latter was useless. All windows, except a few on the fourth floor front, which had iron outside shutters, were protected by wire glass in metal frames, automatically closing. Although some of the mill coping fell, the tank remained, no sprinklers operated. Hose service was useless on account of lack of power, but the employees stood their ground and fought off the exposing fires with pails of water. The first day's fire was on the north and east, and a most severe attack occurred on the south and west on the second day, with very hot exposure on the west, so that cases of ignition occurred through the wire glass windows. The factory was saved and resumed business when the fire passed. It had been only recently erected and was the only plant in San Francisco constructed and equipped up to Factory Standard, corresponding with the standards of the National Booard of Fire Underwriters for self-protecting factories."

In the great Baltimore conflagration of 1904, the new Transformer Station of the Electric Light & Power Co. stood directly in the path of the flames. It was equipped throughout, as was the California Electrical Works, with wire glass windows, and like the latter, came through the ordeal practically undamaged. Such examples of the fire resistive power of properly designed buildings are very encouraging. They show very clearly that it is thoroughly practical to do away with great conflagrations—that window openings may be rendered absolutely proof against the attack of fire by eminently practical and universally applicable means. The wire glass window is never "inadmissible." It may be made to harmonize with any architectural treatment. It may be applied with equal facility to either old or new buildings, and it gives a maximum degree of protection at all times.

Solely by reason of the great conflagrations of the past decade, the business of fire insurance is in a sadly demoralized condition, rendering it difficult, if not impossible, for the average merchant or manufacturer to secure his requisite quota of indemnity. The influence of this fact on business credit is enormous, and the situation has small chance of improvement until owners in general adopt individually, means for protecting their buildings against attack of outside fire.

To sum up briefly, while complete fire protection is in the highest degree desirable, it demands so many conditions as to be seldom obtainable. On the other hand, protection against the exposure hazard—with consequent elimination of great conflagrations—may be obtained at comparatively small cost and in old and new buildings alike.

CONCLUSIONS.

1. Because the shell of a building will not burn, it does not follow that the contents are safe against the attack of fire from without.

2. Conflagrations are the result of the spread of fire from one building to another through, in practically every instance, unprotected window and skylight openings.

3. The most efficient and universally applicable type of window and skylight protection is wire glass in metal framing.

4. The largest degree of fire protection for the least expenditure of time and money is obtainable by a reduction of the exposure fire hazard, as it does not involve the reconstruction of the entire building.

Meetings of Technical Societies

(1) AMERICAN SOCIETY OF CIVIL ENGINEERS.

The annual meeting of the San Francisco Association of members of the American Society of Civil Engineers was held at Tait's Restaurant, San Francisco, Wednesday evening, December 19, 1906. The meeting was preceded by a dinner served at 6 p. m.

Officers were chosen for the coming year as follows: President, A. L. Adams; one Vice-President, C. Derleth, Jr.; Secretary, F. Riffle; Treasurer, J. D. Galloway.

The members were informed that the various reports of sub-committees relating to earthquake and fire studies had been transmitted to the parent association in New York. These reports were accompanied by a letter of transmittal and general summary of conclusions arranged by the General and Executive Committee of the Association. This general report was read by the Secretary.

The remainder of the evening was devoted to an interesting discussion of the following papers published in the September and October numbers of the Proceedings of the American Society of Civil Engineers: "Additional Information on the Durability of Wooden Stave Pipe," by Arthur L. Adams, M. Am. Soc. C. E.; "Recent Practice in Hydraulic Fill Dam Construction," by James D. Schuyler, M. Am. Soc. C. E.

(2) STRUCTURAL ASSOCIATION OF SAN FRANCISCO

Since our last reference to the work of this society, three regular meetings have been held at the Mechanics' Institute, San Francisco, as follows:

November 15th: Progress Report on the Tests of Brick versus Stone Concrete, by L. E. Hunt; with discussion. December 5th: Report on the Collapse of the Bixby Hotel at Long Beach, California, by Lewis A. Hicks; Final Report of the Committee on Fireproofing of Steel, Partitions and Interior Finish, F. H. Meyer, chairman. Considerable discussion followed the reading of these papers. December 20th: Paper by Mr. J. D. Galloway entitled "The San Francisco Building Ordinance"; paper by Mr. F. E. Schmitt, Jr., entitled, "Some Observations on the Lateral Vibration of Tall Buildings"; paper by Charles Derleth, Jr., entitled "Reinforced Concrete Construction; its Proper Application in Earthquake Countries."

It was voted at the last December meeting to discontinue the regular public meetings of the Association and to call meetings after the first of January only, after the final reports of the sub-committees had been received and edited. The final report of the Association printed in volume form, is expected to be published some time during the coming spring.

BULLOCK AND JONES' BUILDING

Second Story, Tile Fire-Proofing

MONADNOCK BUILDING, BASEMENT

Column Never Fire-Proofed: Building Under Construction

(3) TECHNICAL SOCIETY OF THE PACIFIC COAST.

A regular meeting of this society was held on Friday evening, December 7th, at 8 o'clock, at the Mechanics' Institute, San Francisco.

A nominating committee was selected to arrange for the election of officers for the ensuing year.

The evening was devoted to a discussion of the important subject of "Reinforced Concrete Structures."

Mr. Lewis A. Hicks discussed the collapse of the Bixby Hotel at Long Beach, California; Mr. M. C. Couchot read a paper dealing with the situation for reinforced concrete construction in San Francisco; Mr. Charles B. Wing read a comprehensive paper presenting simplified formulas with tables for the calculation of reinforced concrete beams.

Mr. Wing presented tables and diagrams similar in construction to those to be found in handbooks for rolled steel, and argued that a favorable step would be gained in the calculation of reinforced concrete members if tabulations similar to those now used for the properties of rolled steel shapes could be introduced in a parallel way in reinforced concrete designing.

Mr. Charles Derleth, Jr., closed the discussion of the evening by further developing some of the principles presented by Mr. Wing. He argued that advancement in the design of reinforced concrete structures was not so much a matter of the application of formulae in calculation, but rather a practical problem dealing with the design of details and connections. He advised a more careful consideration of the frame work for reinforced concrete structures and emphasized the desirability of a complete and unified frame similar to that for a steel frame building.

Heating and Ventilating and Mechanical Requirements of the San Francisco Building Law

By William E. Leland*

IN considering the question of the heating, ventilating and mechanical requirements of the building laws of San Francisco as revised after the recent earthquake and fire, I am at a loss where to begin; not from the large number of such regulations, but from the very lack of requirements in these branches. Aside from a few stipulations relative to the installing of hot air furnaces and their appurtenances, the only requirements are contained in a few paragraphs.

Section 107 specifies that all stacks shall be constructed of steel, brick or reinforced concrete; and further adds that if of steel, they shall be at least ¼″ thick, this thickness increasing toward the bottom or weight and lateral wind pressure may require. No mention is made of the size of the stack, and in many cases where only a small heating apparatus is installed, a stack of 12″ to 20″ diameter would be sufficiently large. ¼″steel for the upper sections of a stack of this size is extremely heavy, and I should consider that in a case of this kind ⅛″ iron was more than thick enough for the purpose. In most cases, too, the stack would be in a vent shaft or light well, where wind pressure would not count, and where it may be supported at intervals in its total length.

In the San Francisco Post Office, which represents good, conservative practice, the stack is 48″ inside diameter, 95′ high and is built of ¼″ iron for the upper 50 feet and ⅜″ iron for the lower sections.

It would appear that the size of stack should be one of the governing conditions in the building law, and that some formula giving a relation between thicknesses of metal and diameter of stack would be much more suitable than a definite thickness for all sizes.

An empirical formula which gives very fair results may be obtained from the regular formula for the thickness of cylinders, assuming a working fibre stress of 2,000 lbs. per square inch and an internal pressure of from 20 to 40 pounds, according as the section in question is at the top or bottom of the stack. The formula would then be $t \quad \frac{p e r}{2000}$ in which it will be the thickness required, **p,** the assumed pressure, and **r,** the radius of the stack. The assumed fibre stress of 2,000 lbs. takes into consideration the fact that the efficiency of

TELEPHONE COMPANY'S BUILDING

Hyde Street, Near Sutter

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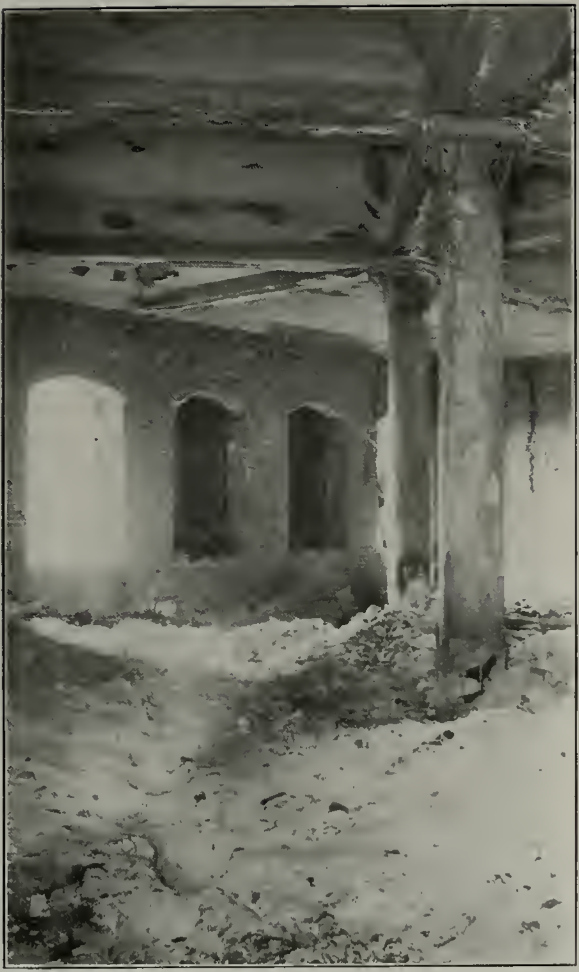
*A paper read before the Structural Association of San Francisco, Meeting of September 27, 1906.



REAR VIEW, INTERIOR ACADEMY OF SCIENCES BUILDING

Showing Kinnear Shutters

the riveted joint in this class of work would be not over 50 per cent, and allows a factor of safety of about 12, which is amply large to allow for corrosion by the elements.

While considering the question of stacks, I wish to call attention to a new material which has recently come to my notice. It is called asbestos lumber, and is composed of cement and asbestos fibre pressed into sheets while wet. In using it for stacks it is rolled into shape and riveted with aluminum rivets and washers in the same manner as an iron stack.

The material is fireproof, acid proof, and weather proof, and is besides a very good non-conductor of heat. It is very hard and tough and is not easily broken. It is much lighter than iron, and to my mind is an ideal material for smoke stacks. It is also ideal for use in the place of corrugated iron for siding and roofs.

It should certainly be recognized in the building law as permissible for use in this connection.

The only mechanical appliance required in the whole building law (outside of elevators) is the arrangement for ventilating the stage of theaters in case of fire. This section requires the opening, by means of a cord, or chain and lever, etc., of two 36-inch ventilators in the roof of the stage. This arrangement is very good, and if the construction and installation is such that it will always work when needed, it should be quite effective. The damper is to be operated from the stage, also from the box-office, and a very nice adjustment of chains, pulleys and levers must be had to make the apparatus positive.

All boilers for the use of heating apparatus, etc., in theaters are required to be placed entirely without the building and in a room built with masonry walls, fire proof floors and ceilings.

I see no necessity for this requirement and believe that low pressure heating boilers could, with perfect safety, be placed under the stage; of course, in a fire proof room. In any large city, in the location in which theaters are desirable, space is usually very valuable, and to require an owner to set apart a space of 300 to 500 square feet for a boiler room entirely outside his building, except in a case where a whole power plant is required, seems quite like a considerable hardship, without a compensating gain in safety to the public.

An exceedingly good situation is found in section 221 in which wall chases for water or other pipes must be filled up for a distance of one foot above and below the floor construction with incombustible material; thus preventing the possibility of a chase acting as a flue for the spreading of a fire.

Section 287 requires the protection by metal shields of all woodwork coming near steam or hot water pipes for heating.

A modification of this requirement so as to allow the omission of the metal whenever the pipes in question are covered with any of the standard non-conducting coverings might be an improvement. These coverings are clearly fireproof and really form a much better protection for the wood than the thin metal which is a good conductor of heat. I consider that the fire hazard in the case of a pipe covered with a good non-conducting covering is far less than in the case of a bare pipe with the adjacent woodwork covered with tin. In any case the danger of fire from steam or water heating pipes is exceedingly remote, and I never heard of a fire traceable to this cause.

There appears to be an inconsistency in this section in the matter of hot water pipes. The section distinctly stipulates the above protection for hot water heating pipes and fails to mention domestic hot water pipes. There is as much danger from the latter, in which the temperature of the water is often as high as 180 degrees when good circulation is maintained, as in the case of a heating pipe, in which the water temperature rarely exceeds 180 degrees and can never exceed 212 degrees.

In point of fact there is not the slightest need of protection on any hot water pipe as a temperature of 212 degrees could never, under any condition, cause a fire or even the blackening of any woodwork resting directly against the pipe.

It might be of interest to mention a fact I noticed after the fire in connection with metal ventilating flues.

In the Fairmont Hotel, all inside bathrooms were ventilated by galvanized iron flues of No. 26 iron, 8" in diameter, leading up to the roof, where they were all gathered into mains and led to two large ventilating fans. As a rule, these flues were in groups, near the building columns, and after the removal of all of the interior partitions, furring, etc., I found them undamaged by fire, and in many places the solder did not run from the joints, even where the flues were close to columns which were badly damaged. There was not a single piece of this pipe that was burned off.

The only explanation I can offer is that possibly cold air passing up through the flues from below kept them cool, but, as a rule, the openings in bath rooms were stopped with boards tightly fitted into the register boxes, thus preventing a positive draft until the fire had burned out the wooden stops.

From the standpoint of an engineer I should like to see embodied in the building laws a requirement compelling mechanical ventilation in all audience halls and schools wherein a certain minimum quantity of fresh air should be supplied for each occupant.

In a number of the Eastern states this is compulsory in the case of schools, and it should be here. In the case of theaters it is more of a comfort than anything else, as the audience occupies the theater only about three hours, and there seems to be no good reason to compel the owner to provide for the comfort of his patrons, if he is not so inclined. In the case of schools, however, where our children spend five or six hours every day, ventilation becomes a matter of health, and should be looked upon as a very vital point, even as much as safety from fire or other injury; and a compulsory requirement of this nature seems a necessity. This properly should be a State regulation, but the City could easily be progressive enough to make the requirement itself.

FAIR BUILDING

Front and Pacific Streets, Showing Corrugated Iron Doors

WIND STORMS AND SAN FRANCISCO FIRE RUINS

ON December 10, 1906, an exceptionally violent rain and wind storm visited San Francisco and vicinity, causing considerable discomfort and much damage. Many unsafe fire ruins were partially or completely thrown down. We might mention many cases of such collapses, but refer only to the Justinian Caire front and to the walls of the old Sterling Furniture Company's building. Both these ruins are on Market Street.

In some instances walls collapsed and crushed temporary wooden buildings adjoining them. Fortunately there was little injury to persons. It seems fortunate that more activity is not shown in razing these dangerous fire ruins. A feeling of uneasiness always comes over the writer when he passes along Market Street under the walls of the old Emporium Building. It is a good sign to note that there is some indication of progress toward removing these dangers.

It is not the intention of this journal to merely criticize. We appreciate the difficulties which constantly confront those who are solving San Francisco's problems. It is easy to find fault and there are many things we would like to see done at once for which there is good reason for the delay.

During the past month there certainly has been very substantial progress toward the rehabilitation of the business district. The streets are rapidly becoming more passable. It is to be hoped that lower Market Street will now be raised to final grade. The sooner this work is done the better, and every day of delay will cause more inconvenience. At best, lower Market Street will be congested for several months to come, and it would be better to suffer an increase in that congestion now than at a later date.

Advantages of Conduit Systems Over Knob and Bushing Work; and Circuit Breakers on Service Boards Over Enclosed Fuses

By H S. Tittle *

THE electrical installations will form a very important part in the rebuilding of San Francisco and the advantages of conduit systems over knob and tube work should be considered by all concerned in construction work.

It is more important to the owner than to the architect or engineer that he have a system which will render perfect service at all times, and keep the cost of maintenance and repairs down to the minimum. The only perfect system to be had is the Iron Conduit.

By the installation of this system, the wiring is protected from mechanical injury and the fire risk is reduced to the minimum, to say nothing of the cost of reparis. In knob and tube systems, after the completion of the building, should a fault occur, it is rather difficult to locate and is attended by more or less damage to the building during the process of repairs. In conduit systems the wires are accessible at each outlet and in case of trouble it is a simple matter to locate the section giving the trouble and withdraw the defective parts and replace them with new wire.

If San Francisco's Electrical Ordinance be amended, permitting only conduit systems in all classes of buildings inside the fire limits, this would place the work in line with the best practice and would reduce the unscrupulous contractors' opportunities for improper work The prime reason why conduit systems are not more generally used, is the cost, which is a small item, however, where first class service and good work are required. In class "C" buildings, the labor cost on conduit systems can be materially lessened and a better system obtained by cross-furring the ceilings with ⅞" strips. This will permit of the standard ½" conduit being strapped to the under side of the joists and changes in direction could be made by long, easy sweeps instead of 90 degree turns. The cutting of joists would be practically eliminated, which is worthy of consideration. By making long, easy sweeps instead of 90 degree turns, the process of pulling in wires is greatly facilitated.

The conduit systems in the Mutual Savings Bank Building, the Shreve Building, Grant Building, and St. Francis Hotel Building, were in comparatively good condition after the fire, the only portions damaged to such an extent that replacing was found necessary being the drops from the ceiling line to bracket and receptacle outlets. All piping behind metal lathing in the ceilings was found to be in fairly good order, it being necessary only to remove the wire from the ceiling runs.

The question of proper protection on service boards should be considered by those interested

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*A paper presented to the Structural Association of San Francisco; Meeting of September 27, 1906.

FIRE EFFECT ON TILE FIRE PROOFING

in construction work. This is the point where the device known as the "Circuit Breaker" has its advantages over the present method of cartridge fuses.

It is only a short time since the National Board of Fire Underwriters condemned the skeleton service boards constructed of wood, on which were mounted porcelain fuse blocks with open lead alloy fuse wire. From the skeleton wood frame they passed to marble panels, still retaining the open lead alloy wire fuse. The last issue of the National Electric Code has condemned the use of open wire fuses on porcelain bases, as well as on marble service panels where the fuse wire is exposed. They have adopted instead the "Non-Arcing Enclosed Type Fuse," the lead fuse wire being enclosed in a small fibre cartridge. All of these were steps in the right direction. They should go one step further and condemn the uses of fuses on service boards entirely, adapting as a substitute the Circuit Breaker, which is positive in action and protects the circuit by opening it automatically.

Many times on service boards, when a fuse blows, if it be the enclosed type and no fuses are at hand, to replace it, the attendant, in order to close the circuit, looks around for any sort of metal and places the strip in the circuit without reference to its carrying capacity. By this act the purpose of the fuse is defeated.

Many architects in preparing specifications relating to service boards will specify double throw switches, with fuses on both sides, so that in the event of a short circuit occurring, the switch may be thrown from fuse No. 1 to No. 2. This, however, is not good practice, for, should the short circuit which blew fuse No. 1 remain on the circuit, the act of throwing the switch over to No. 2 merely blows that one also, or if fuse No. 2 be defective and permit current enough to raise the temperature of the wire in the circuit to the danger point, a fire might be the result.

Under similar conditions, if circuit breakers were installed, the short would merely cause an opening of the circuit which could not be restored until the fault was located and repaired.

The Westinghouse Electric & Manufacturing Company have recently placed on the market a small Single Pole Circuit Breaker which is positive in action and nominal in cost.

Tablet Boards and their installation are worthy of some consideration. The National Electric Code states that tablet boards located in walls, where the back is not accessible, must be front connected, the marble side linings of the cabinet enclosing the tablet to be set so as to form a gutter around the tablet to permit of the wires being carried to their proper position. This method makes it necessary to drill holes in either the side linings or through the tablet to permit the wires being connected to the switch and fuses.

It would be better practice to locate all distributing centers in partitions where the back of the tablet would be accessible. This would mean a through opening in the partition with a neat and panel door on both sides. The tablet could then be made back-connected, so that the only parts showing on the face of the board would be the switch and fuse. The busbars and wire connections would be on the back and readily accessible for all renewals and repairs.

NOTES ON BRICK WORK

By P. J. Walker*

AFTER the smoke incident to our unfortunate disaster of some months ago had cleared away, and the sombre spectacle of an isolated spire of masonry here and there presented itself, the prophecy was freely made that from that time forth, San Francisco would witness a complete revolution in the manner of securing proper masonry work and in the execution of the same. In other words, the first thing that became apparent at that time was that more attention would have to be paid to the future construction of brick walls.

Unfortunately, I am obliged to confess that so far, with very few exceptions, this disastrously learned lesson has been absolutely disregarded, for to-day, there is no doubt in my mind but that poorer brick work is being done than before the fire.

There are to my notion, three general reasons for this condition:

First.—The indiscriminate use of old bricks.

Second.—The presence of some unscrupulous contractors.

Third.—The tendency of owners to try to utilize as far as possible parts of the walls left standing.

The use of old brick should not, in my judgment, be entirely prohibited, but many thousands of old brick are being incorporated in buildings at present, which ought to be condemned, and I am glad to say that some of the building inspectors are using good judgment in condemning them. In addition to this, the method of incorporating these old bricks into new masonry should be very closely watched. The brick should be more than ordinarily well wetted; the mortar more than ordinarily strong, and the workmanship, in solidly rubbing up the work, absolutely insisted upon.

I think it would be advisable, as far as possible, to make vent and smoke flues independent of the walls, thereby making it possible to run continuous bond iron in the center of the wall from one end of the building to the other. With the insertion of numerous flues, as at present, it is impossible to place the bond iron where, in my judgment, it ought to be placed, viz.: the center of the wall.

While on the subject of bond iron, I would suggest that instead of letting the joist rest on the iron, as is customary at present, that some other provisions be made for the joist, and the iron placed as before mentioned, in the center of the wall, running continuously from outside to outside, except in places where the face work would be disfigured by the presence of a washer at the end of the iron.

The practice has prevailed here, as well as elsewhere, in building corners of brick work, to tooth-out the brick work in order to reach a given height. This is absolutely faulty construction, for it is impossible to properly bond the remainder of a wall to a corner that has been toothed for several feet. Many times this method of building a corner results in quite a saving to the contractor.

Where beams are made to rest on wall openings, I would further suggest that a longer bearing be given on the wall than has hitherto been the practice, for in many instances the shake has resulted in shearing off the small bearing which has been given the lintel over an opening. Where there are a series of openings covered by lintels, better results would be obtained if it were possible to continuously bond these lintels by means of a strap iron from one to the other. When that is not practicable, the use of larger anchors reaching further into the masonry would be a suitable substitute.

In the matter of brick cornices, I have in mind one, which while being perhaps one of the heaviest in town, stood the shake very well. In that building, which is the building owned by Messrs. J. A. Folger & Co., at the corner of Spear and Howard Streets, the brick cornice is supported and anchored by means of a skeleton steel frame, running entirely around the two frontages. This practice has prevailed in the use of terra cotta cornices, but has not been generally applied, as it ought to have been, to the use of brick cornices. Were this form of anchoring cornices more freely used, I am satisfied that a brick cornice would stand most any kind of torsion.

The practice of corbeling out at the various floor levels for a bearing for joist, is, in my judgment, a very good practice and might be more freely used with good results.

The present provision of the building ordinance relative to some form of bracing for long, high walls, is a very wise provision and one that is absolutely necessary for the safety of buildings having many openings.

In closing, let me suggest to those whose calling is along the Architectural line, that they try to school the owner to a more liberal use of money in an endeavor to get a first class job of masonry. The work should not necessarily be given to the lowest bidder, for, do the best you can, still some man will be prepared to do a job of brick work for next to nothing.

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†Submitted to the Structural Association of San Francisco; meeting September 20, 1906.

Building News and Trade Notes

WE shall welcome any news notes that our readers may send to us. Notes of contracts awarded, bidding prices, etc., will be especially acceptable. While this journal devotes itself most decidedly to architectural construction, it wishes to reflect also the activities of engineering in its various phases, especially civil engineering in its relation to the design and construction of large and important works. We shall be pleased also from time to time to make statements bringing to the attention of our readers the equipment or development of manufacturing plants of interest and worthy of notice. We shall likewise be glad to refer to manufacturing enterprises and manufactured articles, especially relating to structural products for building work.

A New Terra Cotta and Sewer Pipe Company

THE new plant of the California Brick and Clay Manufacturing Company is located at Antioch, California, on the Southern Pacific Railroad at its point of intersection by the right of way of the Empire State Railroad. With a thorough equipment of modern machinery for the manufacture of sewer pipe, chimney pipe, flue lining, electric conduit tile and other clay products, this plant began active operation during the Fall of 1906. Construction work, however, still proceeds and will continue throughout the winter. By next spring this company will be one of the large producers of hollow clay products for the California market.

The plant consists of a large new building 100 x 170′ (see illustration), three stories in height, containing the dry floors. Two wings are now being added to this structure to give additional floor space.

The machinery room is 70′ x 70′ and contains the various pan mills, elevators and bins.

The boilers and engine are housed in a separate two story building 50′ x 85′ which also contains the dry floor for elbows, curves, traps, chimney tops, etc., for the burning of which a special kiln has been provided. A part of the upper floor of this building has been set aside as a molding room.

There are three large kilns for sewer pipe, chimney pipe and kindred materials. They are circular down draft kilns, 24′ in diameter constructed with the purpose of installing a waste heat dryer system which will be in operation throughout the winter. The kiln capacity of the plant is to be increased by the construction of three more 26′ kilns as rapidly as possible and

PLANT OF CALIFORNIA BRICK AND CLAY MANUFACTURING COMPANY
Antioch California

CALIFORNIA BRICK AND CLAY MANUFACTURING COMPANY
Antioch, California

a new two story building will also be erected with a dry floor area of 35,000 sq. feet.

Dry sheds for storing clay are now in course of construction and will have a capacity of 1500 tons.

This Antioch sewer pipe works is most advantageously situated with regard to its raw material. The clay is taken from the foothills of the Mt. Diablo range, is but a short haul to the factory and will descend by gravity almost to the door. The face upon which the company is now working is about forty feet in vertical height, pure white and dove-gray clay. There is no expense of stripping as the pure clay extends to the very grass roots. The California Brick & Clay Mfg. Co., practically controls an unlimited quantity of this excellent material. In sewer pipe it burns to a hard, strong, dark, perfectly vitrified body and takes a practicularly fine salt glaze. Blended with certain other clays owned by the company, a buff facing brick is produced that for strength and vivid yellow color is a very desirable product. It is the intention of the owners of the Antioch Brick & Clay Mfg. Co. to erect a brick kiln and install a face brick plant to be operated in connection with their pottery.

As soon as the plant at Antioch is fairly in operation the Company will have a well stocked yard in San Francisco; their principal office is in the Monadnock Building, San Francisco.

Mr. W. F. Barnes of San Francisco is the president of the Company; Mr. John G. Treanor is its manager; Mr. E. T. Maple is the Superintendent of the plant.

Since this article was originally arranged the Company has begun the construction of an additional building 300′ long and 75′ wide, to be specially adapted to dry-flooring facilities; additional kilns are also being built.

The two accompanying pictures give views of this growing plant.

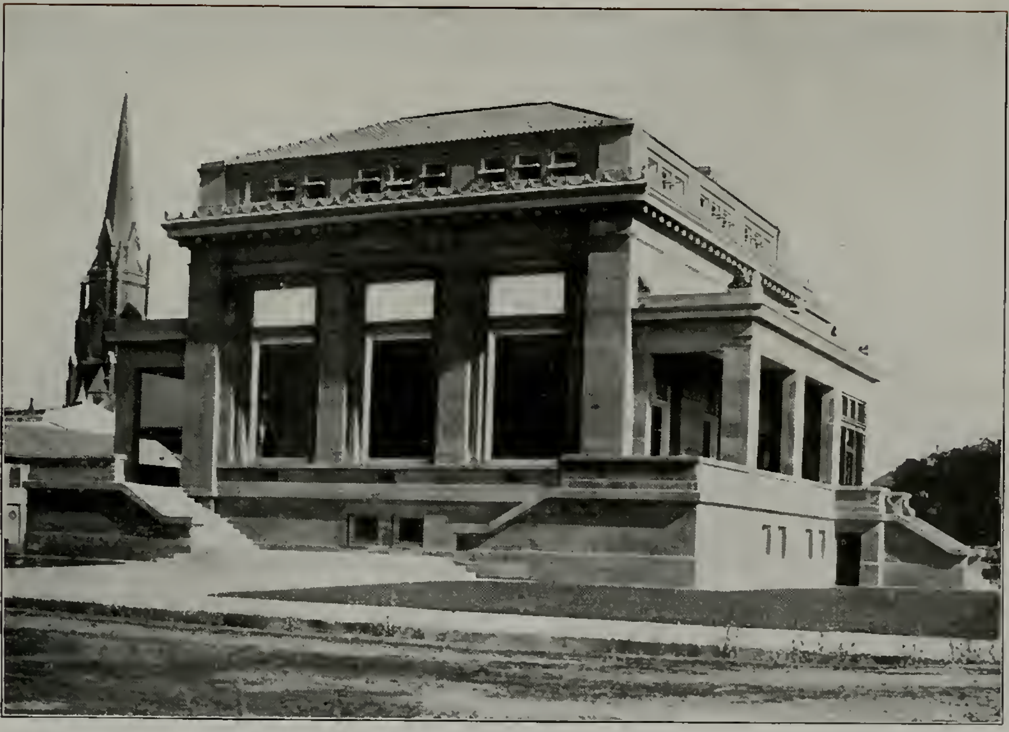
ERRATA

In our December issue, page 105, we printed an article headed, "Metal Paints," in which reference was made to the products of the Ironite Chemical Company, of which the Johnson-Locke Mercantile Company of San Francisco are the sole agents.

A typographical error appears in this article in the fourth line of the last paragraph where the word Ironite should read Iroline. We regret that this error has occurred.

AMERICAN BVILDERS REVIEW

SAN FRANCISCO, FEBRUARY, 1907



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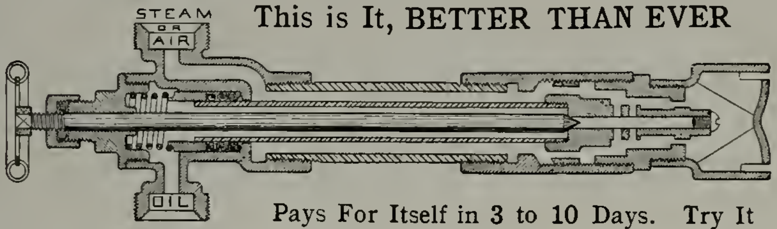

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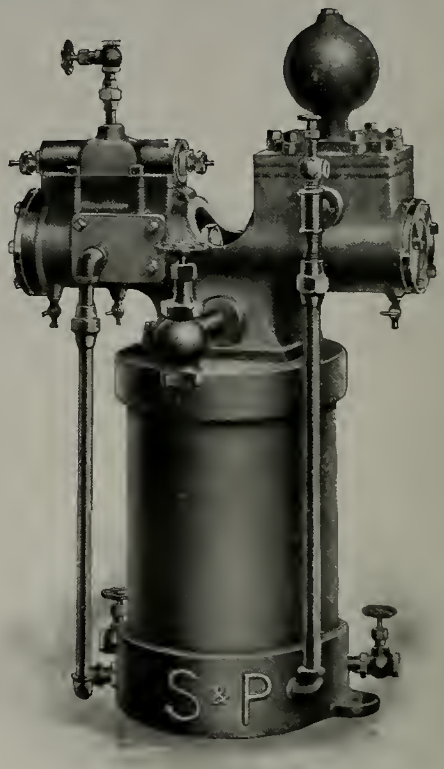

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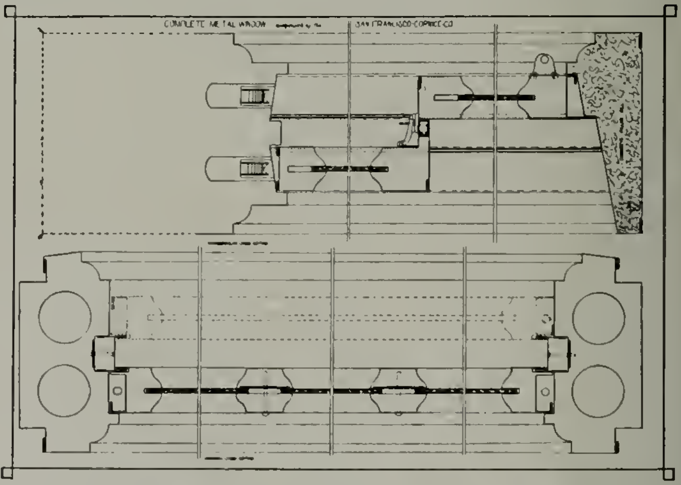

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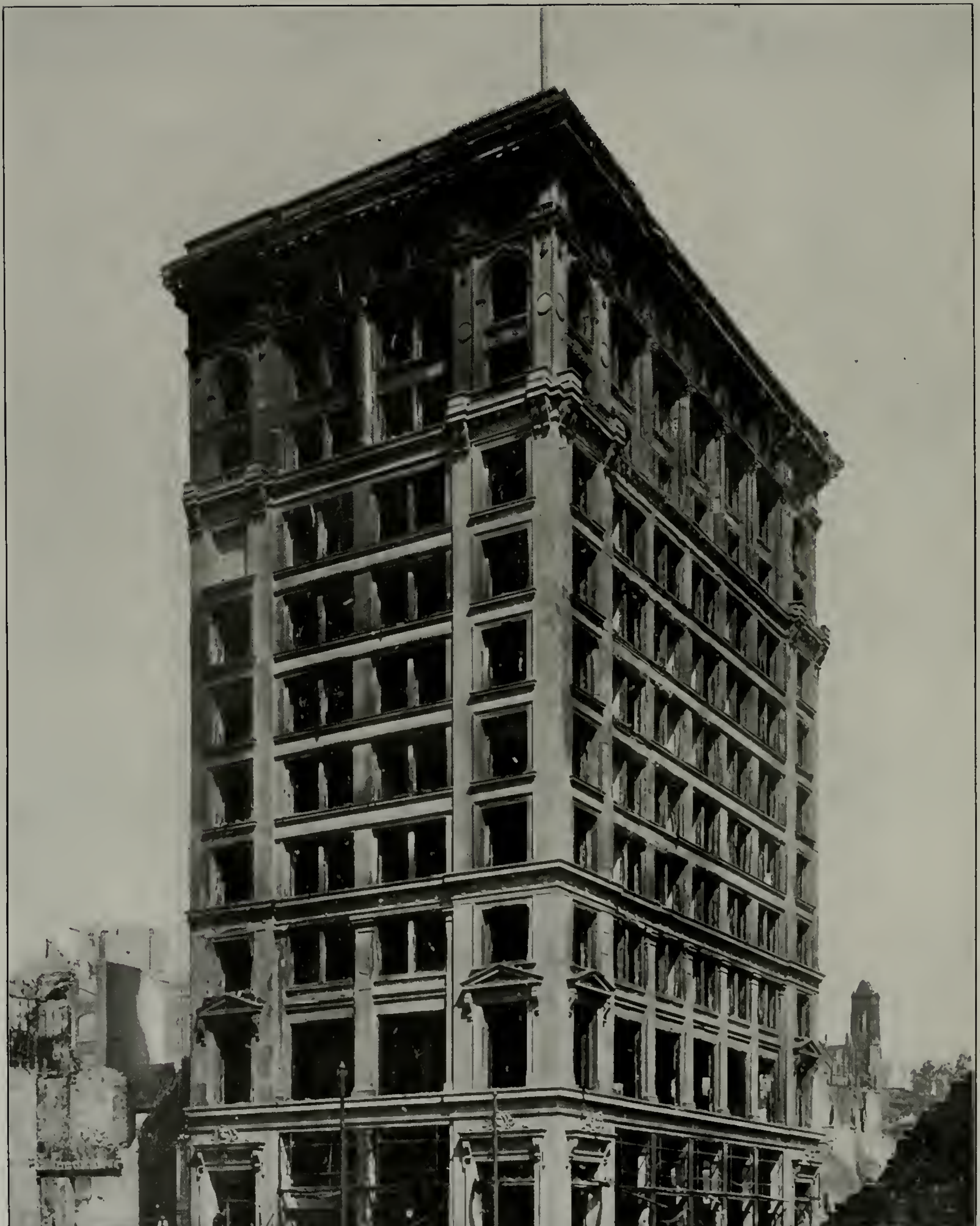


THE SHREVE BUILDING
San Francisco

AMERICAN BUILDERS REVIEW

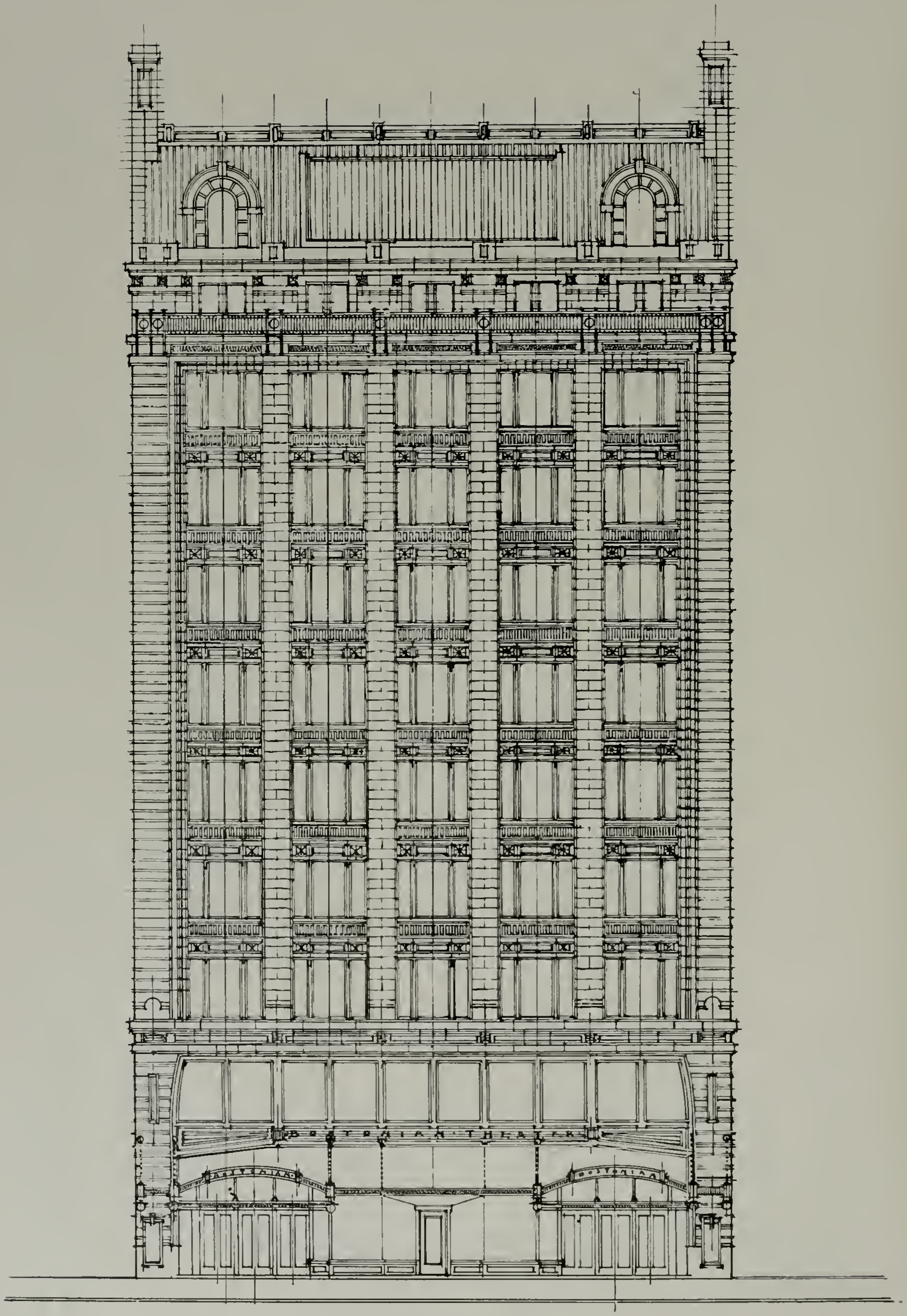
VOLUME VI FEBRUARY, 1907 NUMBER 2

A Theatre and Office Building

By T. H. Skinner

THE drawings herewith presented illustrate two different studies recently submitted by our office to the promoters of a theatre and office building, for up-town Boston. The lot secured was narrow and deep and not ideal for the purpose. Scheme A shows the best we could do, keeping as close to the promoters' programme as possible. A small store in the center was demanded for candy and cigar stands; this divided the exits from the theatre, and pushed the elevators and stairs serving the office building into the rear of the store, cramping the depth of foyer in front of the house very seriously, besides putting these in an awkward and inconvenient position, and making it necessary to use double entrance elevators. The entrance had moreover the disadvantage of not being distinct from that of the theatre. The width required by law for exits left us only room enough for the side walls and the light division walls between lobbies and gallery exits, and made the problem of a facade very difficult. So difficult that we changed the direct entrances from the gallery stairs shown on the first floor plan to that shown on the marginal sketch, so as to give some piers on the corners. The absence of practically all visible supports except at the corners on the street level is masked by the marquise carried across the entire facade. The slightly arched girder and metal mullioned frames of the mezzanine story would, we hoped, satisfy the longing for some visual support for the piers of the upper stories, which is sometimes lacking. In the stories occupied by offices there is nothing of special interest in plan. The facade is a frank expression of the steel frame with light all metal bays between the masonry covering the steel columns. No heavy cornice is planned, but rather a light balcony with a little attic story behind topped with a mansard, showing a large studio light. The house and stage are planned along quite conventional lines and are very little different from those of Scheme B, which is greater in depth from the proscenium, to the foyer.

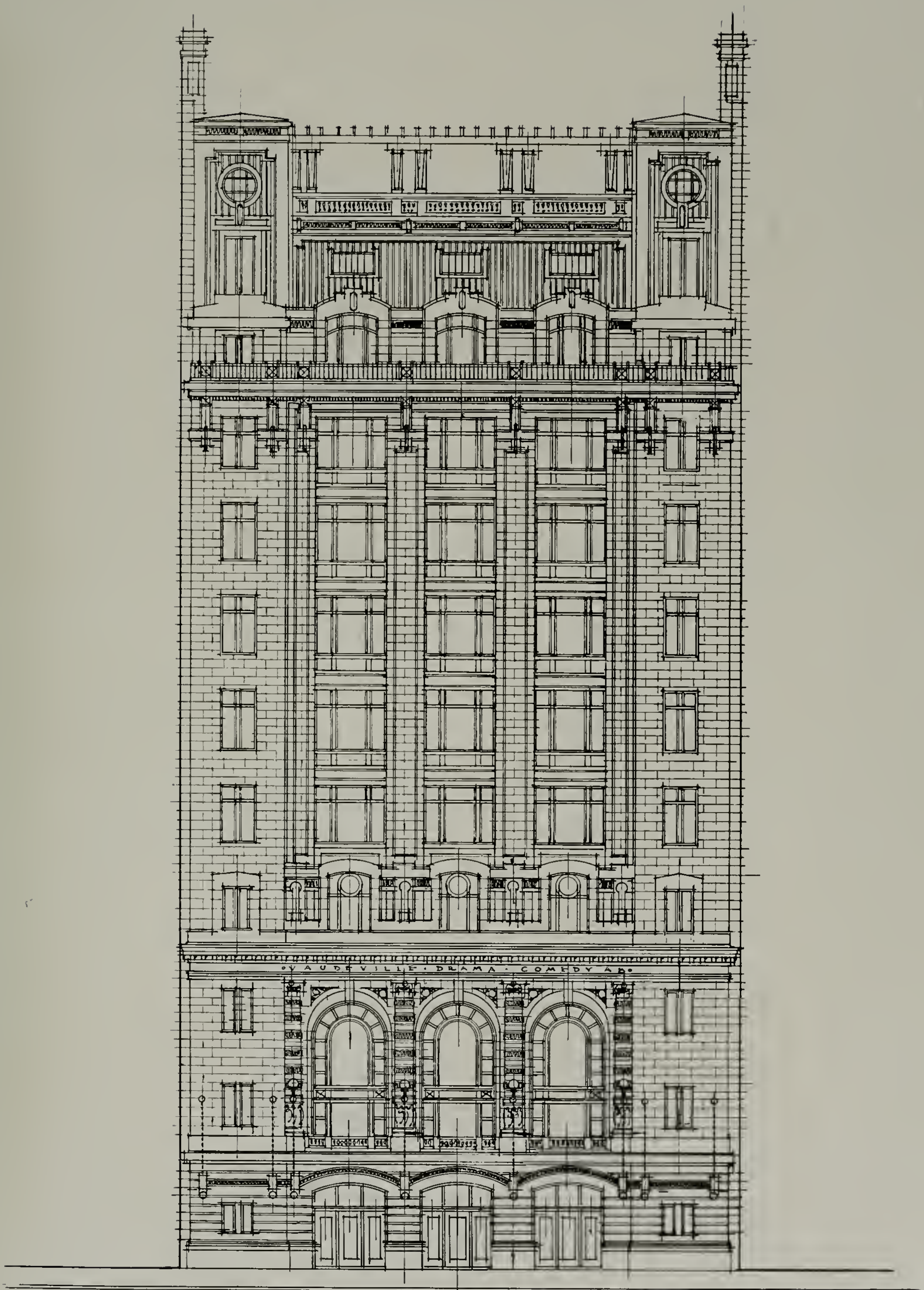
Scheme B is radically different, however, from Scheme A in many ways, and represents we felt much more the architectural character which such a building should have. It has greater solidity and dignity and greater richness of detail. Removing the little central store, made possible, first, a separate entrance to the office building—easily discovered—a finer foyer for the house on the street level, equal if not better exits from floor, balcony and gallery, a larger floor to the house and last but not least some solid masonry piers and walls in the lower stories. The other changes in the office floor plans speak for themselves, and make for a better building. The addition of a roof garden and cafe to the scheme made possible a little more interesting roof line than that shown in Scheme A. The two-storied order, with the arched windows, the enriched entablature, the balconies, etc., of the lower stories of the facade express to us the public character and scale of the house behind them, and make also a substantial foundation for the many storied walls above. The larger seating capacity of the house in Scheme B more than offsets the rental lost by omitting the store of Scheme A, and justifies us in presenting the studies of a scheme directly opposed to the first ideas of the promoters.



Putnam & Cox, Architects

PROJECTED THEATRE
Boylston Street, Boston

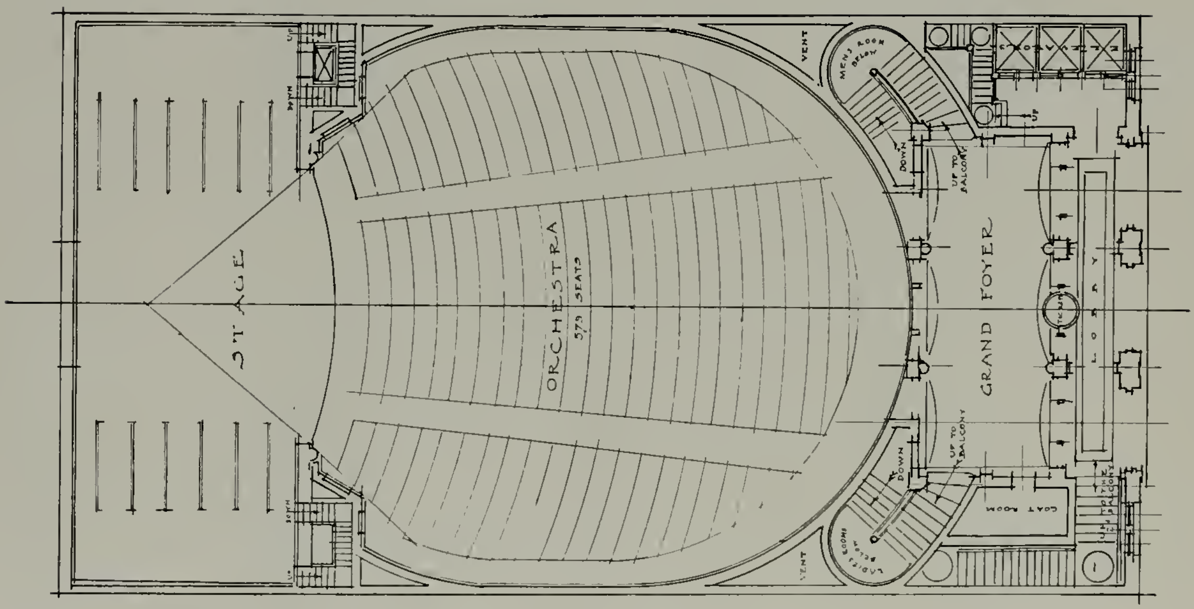
Design A



Putnam & Cox, Architects

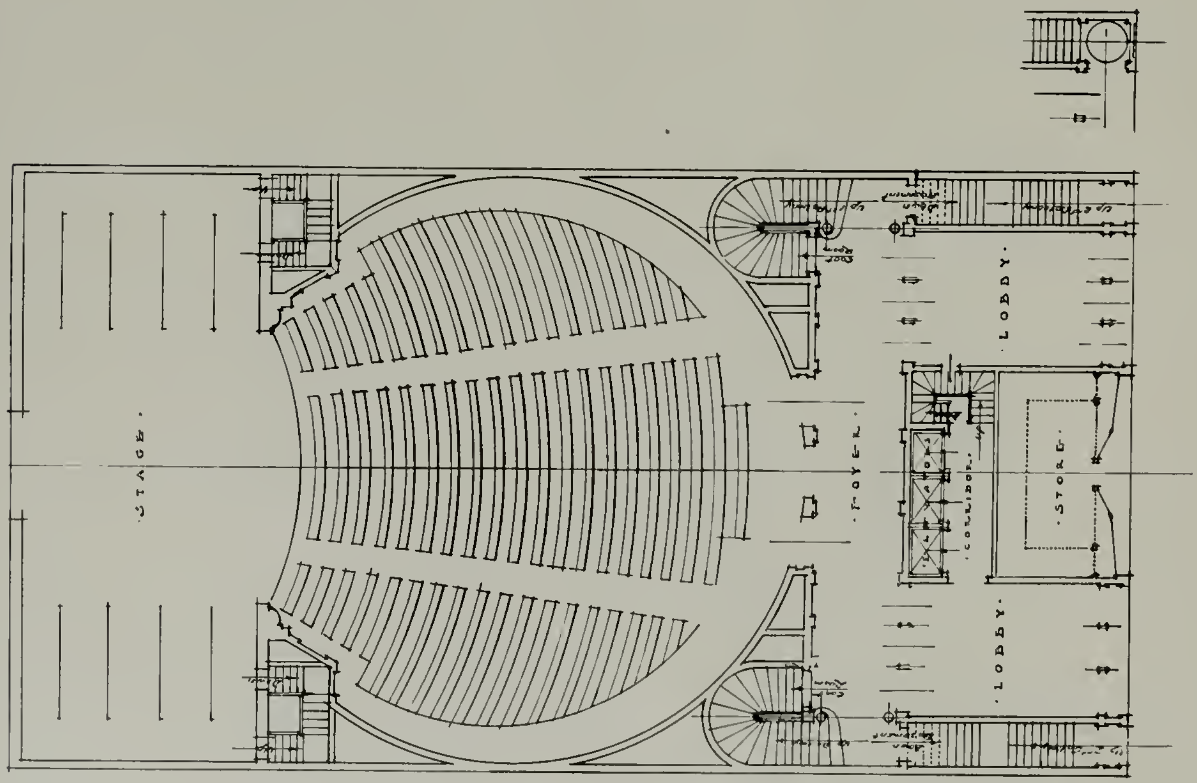
PROJECTED THEATER
Boylston Street, Boston

Design B



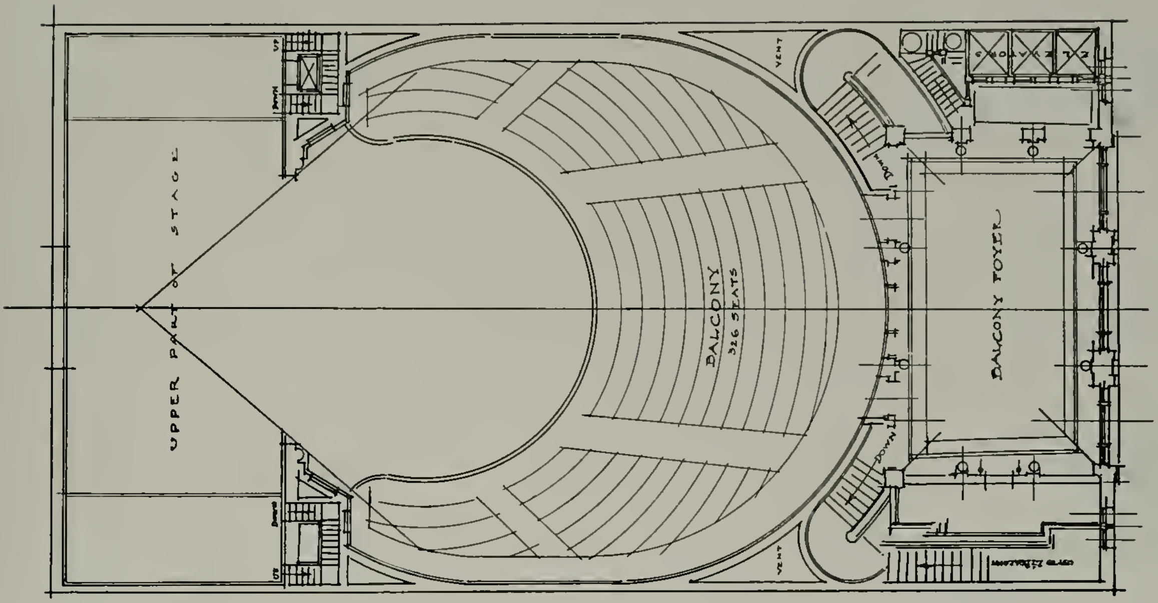
Design B

FIRST FLOOR PLAN



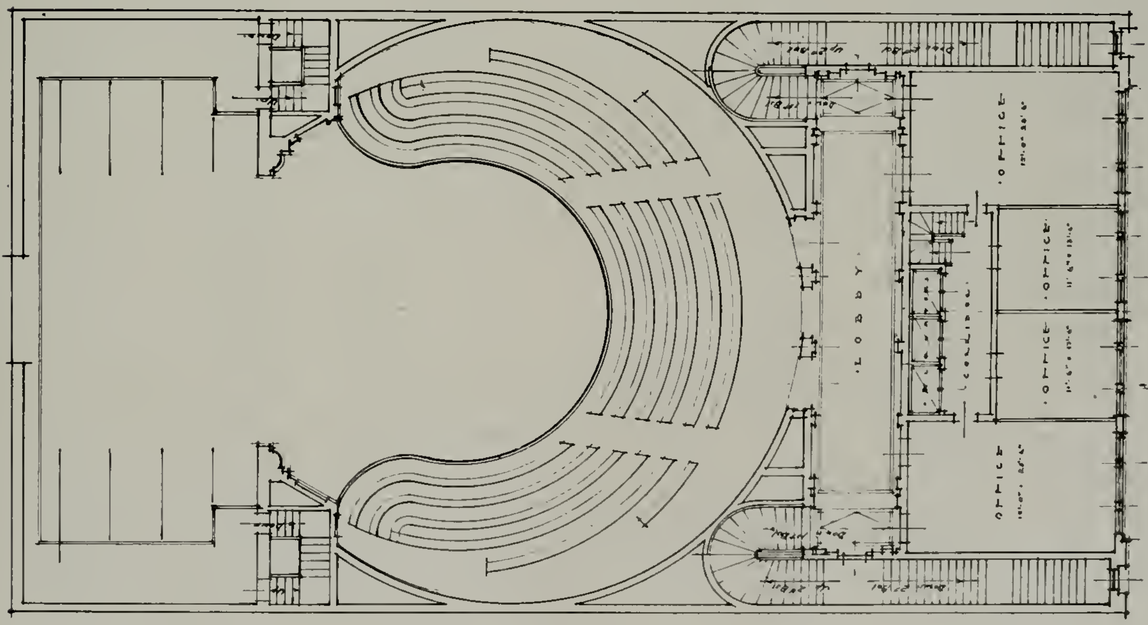
Design A

FIRST FLOOR PLAN



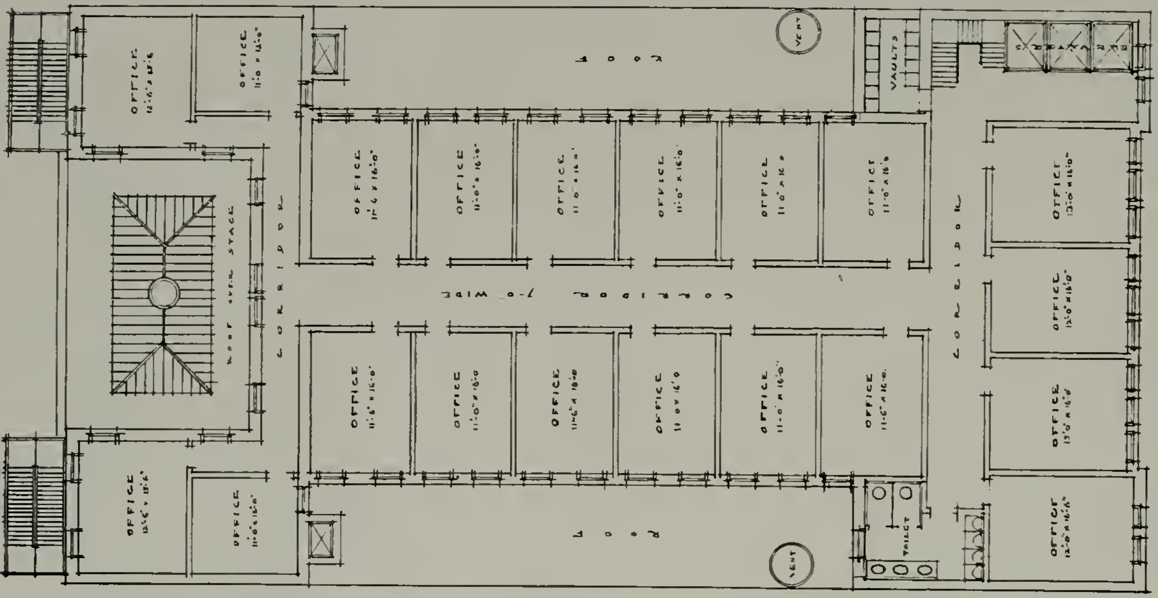
PLAN OF BALCONY

Design B



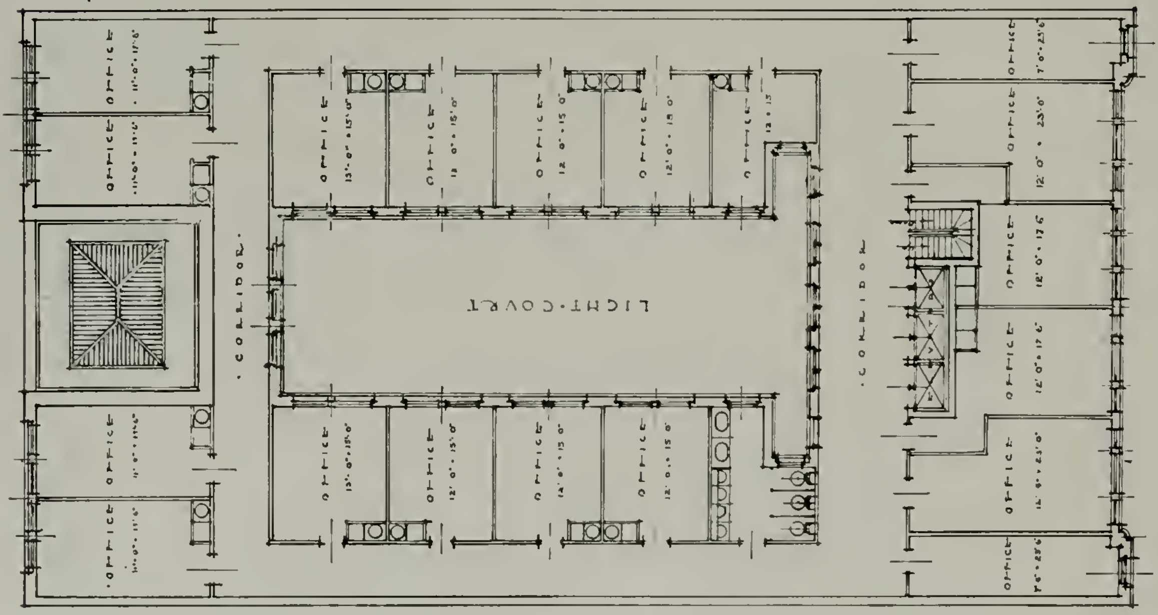
PLAN OF BALCONY

Design A



Design B

TYPICAL OFFICE FLOOR



Design A

TYPICAL OFFICE FLOOR

Wharves of San Francisco

By Carl Uhlig

AFTER testing all patented and unpatented preservatives, with doubtful results, on piles supporting wharves and being exposed to the ravages of marine insects which invade the waters of the San Francisco Bay, the Board of State Harbor Commissioners finally decided to test a cylinder pier wharf designed by the Engineers of that Commission and in 1894 the contract for such a wharf was given out.

The entire load of the wharf is carried by piles driven in clusters of three piles each; a steel cylinder made of 3-16 inch steel plates, 4 feet in diameter, is sunk over each of these clusters; the water and soft mud between cylinder and piles is removed and the space filled with concrete. The superstructure of this wharf consists of wooden caps, stringers and planks as in the older wharf construction.

The durability of this structure as far as the underpinning is concerned, became soon apparent, and a second wharf was, shortly after the completion of the first one, ordered to be built on practically the same lines.

The objection made to these wharves was the great cost of the steel cylinders; further, that during the execution of the work, several of the thin steel cylinders collapsed while being sunk into proper place or being pumped out.

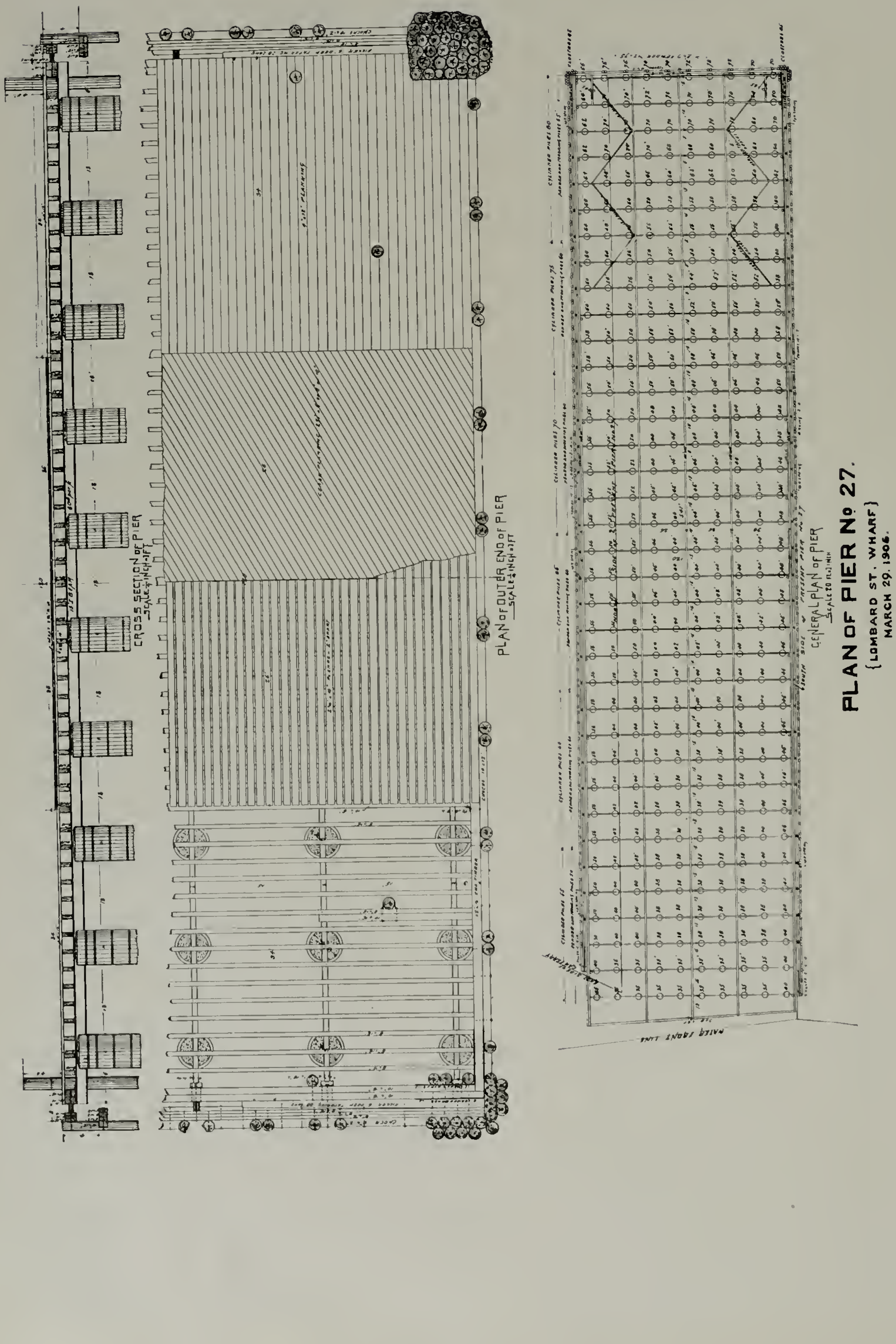
Mr. Howard C. Holmes and the writer designed then a wharf for which they received a United States patent on April 3, 1900, and which has been used ever since on the water front of San Francisco with most gratifying results. The construction of such a wharf is as follows:

The entire load of the wharf is carried by single or bunched piles, the latter driven in clusters according to the loads imposed thereon. These piles are concentrically surrounded with a wooden stave instead of a steel cylinder. The cylinder is considered nothing more than a miniature cofferdam and a form for the pile-protecting concrete and may be destroyed after the setting of the concrete. These wooden stave cylinders have an inner diameter of from 20 to 24 inches larger than the encircled pile or cluster of piles, and are made so rigid and strong that they can be driven with a pile-driving hammer. They are generally driven into the mud to a depth of from 10 to 20 feet, according to its consistency. After being sunk to a stable bottom, the water and mud is removed to a depth of from 5 to 6 feet below the mud or proposed dredging line and concrete, in sacks for the bottom and then loose concrete for the top portion is dumped into the cylinders to completely fill the space between their sides and the enclosed pile or piles. The portion filled with loose concrete is reinforced with an expanded metal or wire mesh cylinder having a diameter of about 12 inches greater than the diameter of the pile or cluster of piles.

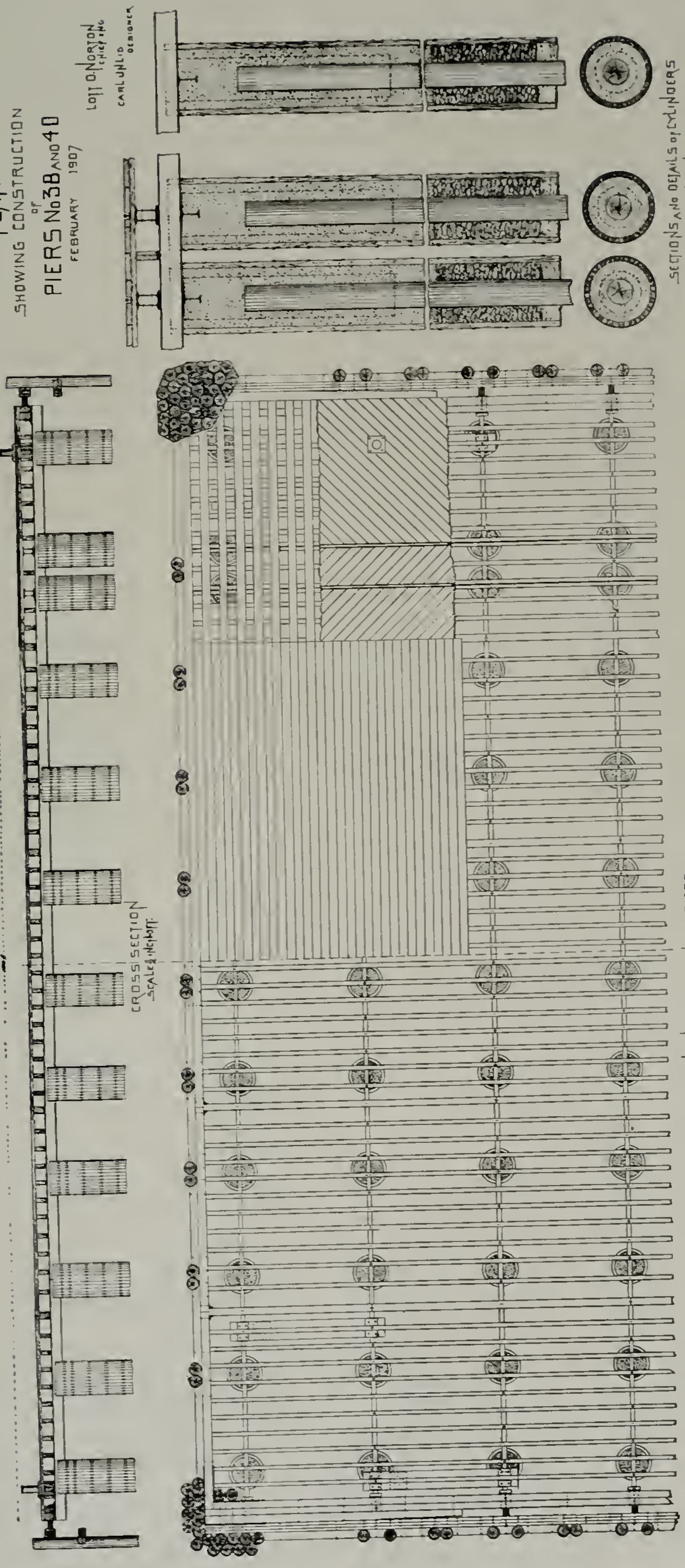
To avoid any severe shocks from large vessels lying against such a wharf in stormy weather, fender piles are set 12 inches outwardly from the sides of the wharf; these are connected by two lines of wooden ribbing and between the upper ribbing and the outer stringers of the wharf proper car springs are inserted to make the spring line elastic and to form a connection between the fender line and the wharf.

Fender lines constructed in this manner have proved very satisfactory not alone as shock preventers but also by giving greater durability to the individual fender piles. In former constructions of wharves, the fender piles were driven and fastened against the sides of the wharves, and being more rigid, they would break off easily in stormy weather whereas in this elastic fender line the piles remain until eaten off by the marine insects.

Besides the great difference in cost of the steel cylinders and wooden stave cylinders, which latter are, even at the present high price of lumber, from 70 to 75 per cent less in cost, the wooden stave cylinders have other advantages, consisting of a larger bearing surface and greater adhesion to the mud; furthermore, materials for them can be brought to the place of work, put there together and after being completed, may be thrown overboard so as not to obstruct the generally much needed space. An unforeseen change in the necessary



RECENT PIER CONSTRUCTION IN SAN FRANCISCO



PLANS FOR LATEST SAN FRANCISCO PIERS

lengths of cylinders, either way, after the same have been driven, is also very easily accomplished.

On acconnt of the expensive repairing or replacing of wooden caps in wharves, the latest constructed wharves have steel I-beam caps to which the wooden stringers and the planking are fastened.

There are at the present 16 wharves on the water front of San Francisco under the jurisdiction of this Board which have a cylinder pier underpinning, and they are equally as good, if not far superior, to the best constructed wharves on this or the Eastern coast of the United States.

There is still room left for improvements in these wharf constructions, especially in the construction of the superstructure, which ought to be of reinforced concrete, but every body requiring a wharf seems more or less averse to a greater original cost.

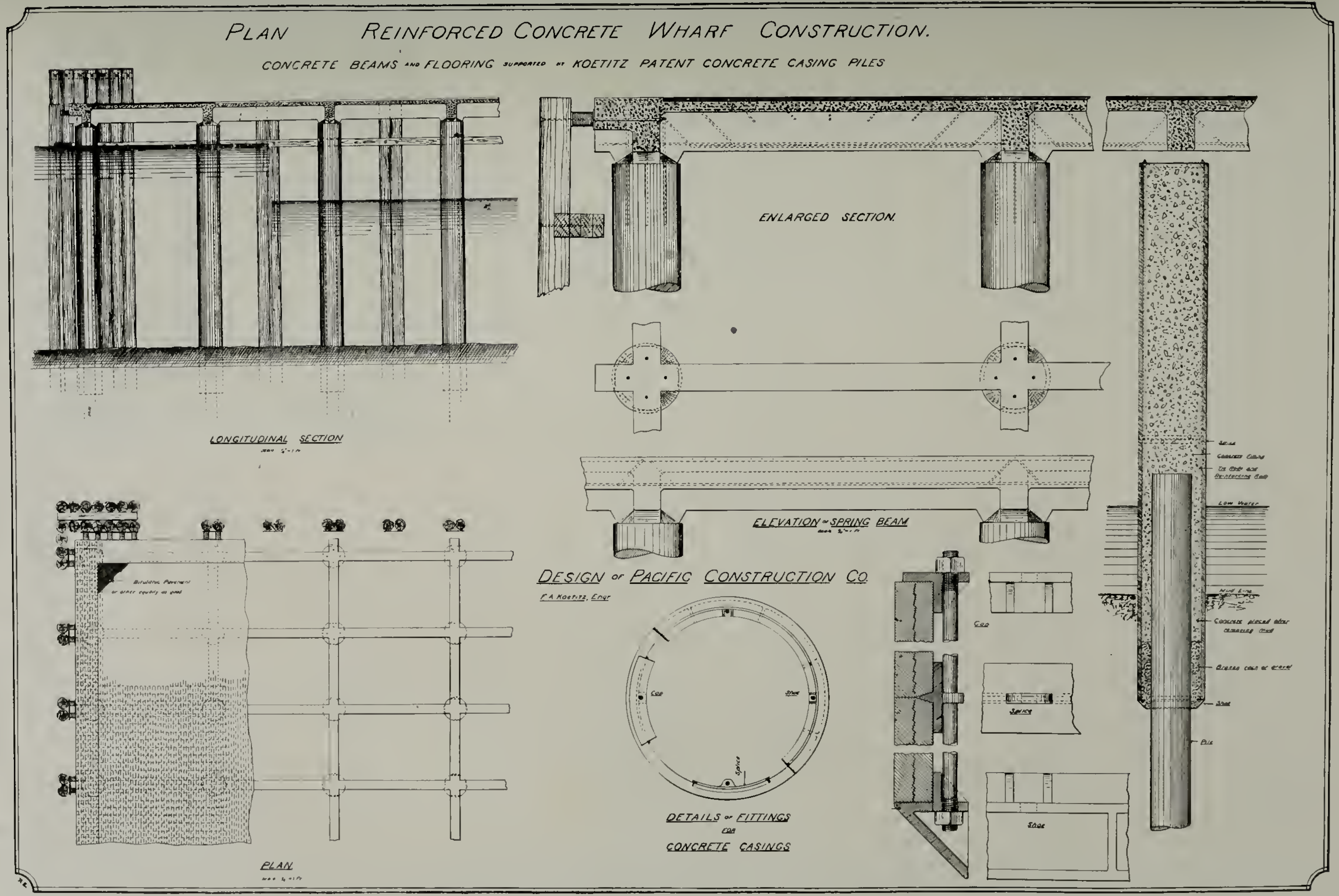
Improvement in Wharf Construction

By F. A. Koetitz

SAN FRANCISCO HARBOR has seen a great variety of wharves. Those built years ago consisted of wooden top work construction on plain wooden piles. The teredo and limnoria devoured these piles so eagerly that many pile preservatives were introduced, the best of them giving a much longer life to the piles. As the loading on the wharves increased, the designs developed into the use of clusters of piles surrounded by cylinders of either steel or wood filled with concrete and the wooden top work was partially replaced by steel construction. There are quite a number of these excellent wharves in use in this harbor now, and there are few better ones in this country. But the time seems rife for a further improvement, as it is desirable that they should be more permanent so as not to require so many repairs that involve a great expense and the interruption in the use of the wharf. It certainly would be more satisfactory and more economical in the end if a practically permanent structure could be built at reasonable cost. It is the object of this paper to outline a design for what I believe to be such a wharf.

Referring to the general plan, we may say that a wharf consists of a "wharf substructure," a "wharf top work," and a "wharf protection." The substructure of our wharves consists of supports 60 to 85 feet long, only the upper 20 to 35 feet are exposed to the marine insects. For economical construction, they should be spaced from 12 to 20 feet apart; they should be as small as is consistent with safety and reduce the exposed surface against the heavy current and waves, and they should be practically indestructible. A concrete pile or cylinder would seem to be the ideal protection, but on account of the great length required, their use would be impracticable and their cost excessive. The design proposed adheres to the wooden pile, single or in clusters, as the case may require, driven in the usual way to a firm foundation and cut off above low water. A reinforced concrete casing of sufficient length is sunk, or driven around the wooden piles, pumped out, and the interior of it filled with concrete. The concrete facing has an exterior face that is permanent and does not rust off as is the case with iron cylinders, or decay or wear off as wooden cylinders do. The cylinder has the best and strongest reinforced part near the outer diameter and can therefore be made consistently of smaller diameter than any other kind of cylinder of the same capacity. It will be noticed that beside the steel reinforcing of the casing, additional tie or reinforcing rods are placed just inside of the casing, imbedded into the concrete filling.

The wharf top work should be firmly connected to the supports, should be free from decay and rust, should not require painting or continual repairs, and naturally nothing but good reinforced concrete construction would do all that. It is so simple that it is a wonder why it has not been used long ago in conjunction with the other cylinders. It requires nothing but simple beams and floor slabs, used for many years in all kinds of structures, including floors of warehouses and machine shops that receive at least as bad treatment as wharves do. No columns and no tile work is included


PLAN REINFORCED CONCRETE WHARF CONSTRUCTION.
CONCRETE BEAMS AND FLOORING SUPPORTED BY KOETITZ PATENT CONCRETE CASING PILES
LONGITUDINAL SECTION
PLAN
Bitulithic Pavement or other equally as good
ENLARGED SECTION
ELEVATION OF SPRING BEAM
DESIGN OF PACIFIC CONSTRUCTION CO.
F. A. Koetitz, Engr
Cap
Shoe
Splice
DETAILS OF FITTINGS FOR CONCRETE CASINGS
Splice
Concrete Filling
Tie Rods and Re-inforcing Rods
Low Water
Mud Line
Concrete placed after removing mud
Broken rock or gravel
Shoe
Pile

F. A. Koetitz, Engineer

DESIGNS FOR REINFORCED CONCRETE PIERS

in this construction, which has been singled out lately as the cause of a couple of unfortunate wrecks, that have scared a few timid or prejudiced souls. No doubt such work for our city's wharves would receive proper care in design and inspection that would leave no question as to its safety.

The top surface of the concrete could be covered with some elastic pavement properly drained to make it clean, lasting and generally desirable.

The wharf protection would consist of the usual lines of fender piles connected to the wharf with suitable springs.

The cost of such a wharf as outlined would be little more than of those we are building now, probably not more than 15 per cent.

In getting out details for such a structure, some improvements and changes will suggest themselves, but no doubt very soon we shall build such structures which, in connection with a substantial stone or concrete seawall, will form a great improvement for our excellent harbor.

A Few Examples of Reinforced Concrete Buildings in Course of Construction in San Francisco

By M. C. Couchot

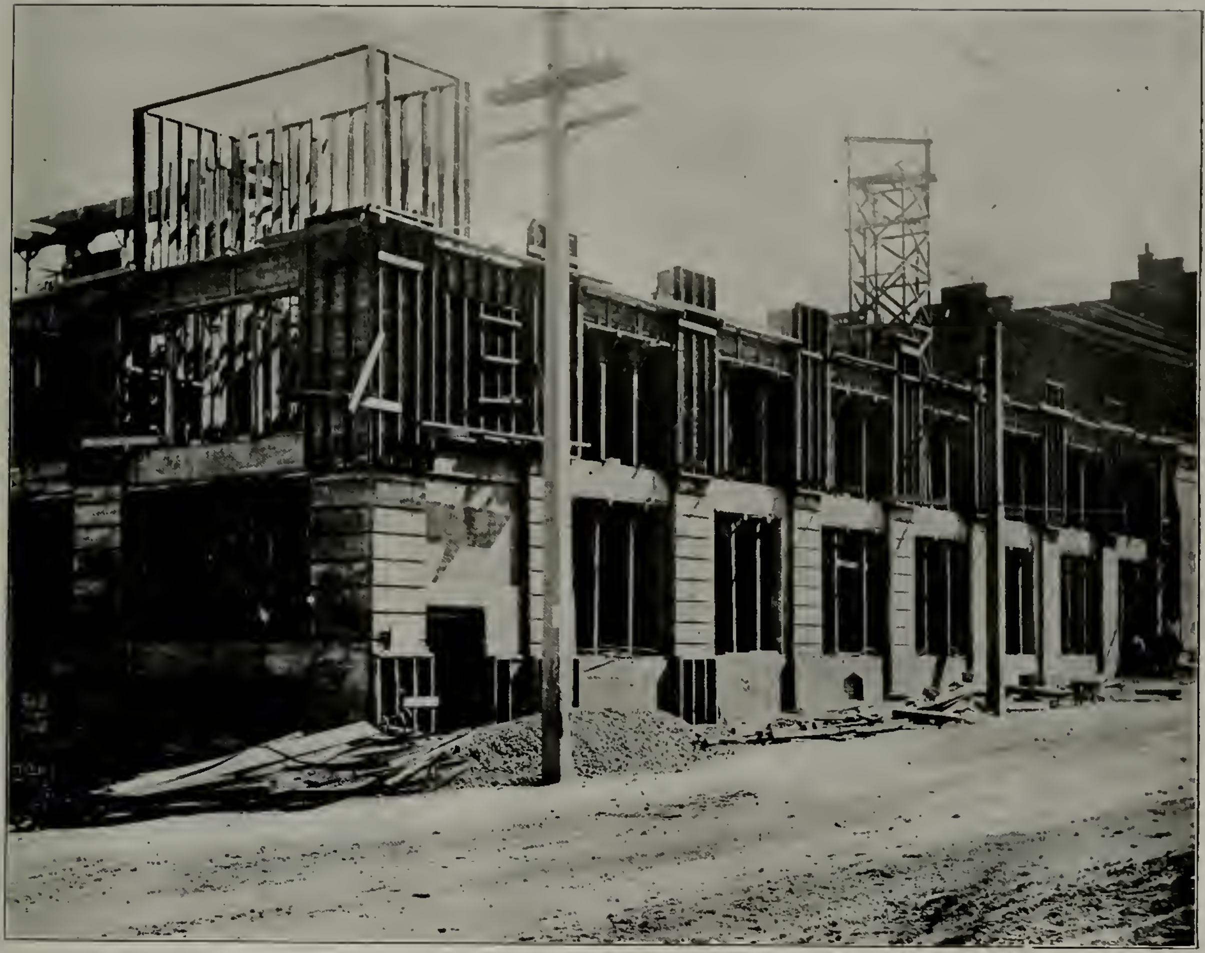
THE restraint put on reinforced concrete construction by opposed interests before the fire of last April had to give way to public opinion at the time of the revision of the Building Ordinances, and in consequence quite a large number of buildings of this type are at present in course of construction in the city of San Francisco. Nearly every system has its representatives among the number. Most all systems [illegible] intelligently designed and careful[illegible]ed with proper materials. Of course, some systems are better, safer and cheaper than others. The writer representing a patented system will limit this description to buildings using that system, namely the Kahn System.

A reinforced concrete building to go completely by that name, must be entirely of concrete; that it, its columns, girders, beams, slabs, walls—in short, every part must be of reinforced concrete, otherwise it is not reinforced concrete at all, although some buildings profess to be which are at present being constructed in the city with a light shell of reinforced concrete for the outer walls and the entire inside a wood construction.

The present conditions in this city are not favoring reinforced concrete construction. Good rock and good sand are scarce, labor inefficient, lumber high in price; cement and steel are about normal.

The first permit used for a building of this construction was obtained for a two-story warehouse 60′ x 75′ on the S. W. corner of Drumm and Washington streets, for Miss Mary L. Phelan, the architect being Mr. Wm. Curlett. The building floor plan is divided into panels of 15′ x 15′ and the floors are designed to carry 250 lbs. live load on the first floor and 200 lbs. live load on the second. The slabs are reinforced both ways. The walls are 8″ thick. It was not deemed necessary to drive piles and a spread foundation was used allowing only one ton per square foot on the soil, for both dead and live load combined. A test made on the site showed no sinking after three weeks under a concentrated load of over two tons

INTERIOR VIEW, FIRST FLOOR
Bemis Brothers Building



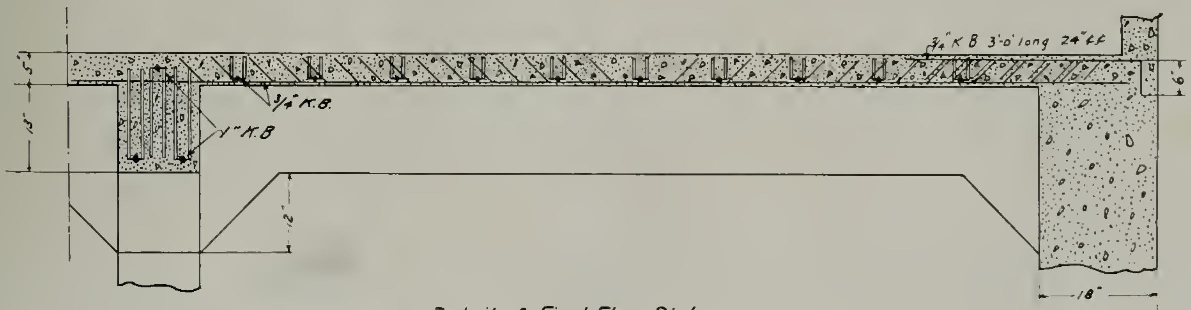
BEMIS BROTHERS BAG FACTORY

Six Story Reinforced Concrete Warehouse

on a single square foot. The building is now completed.

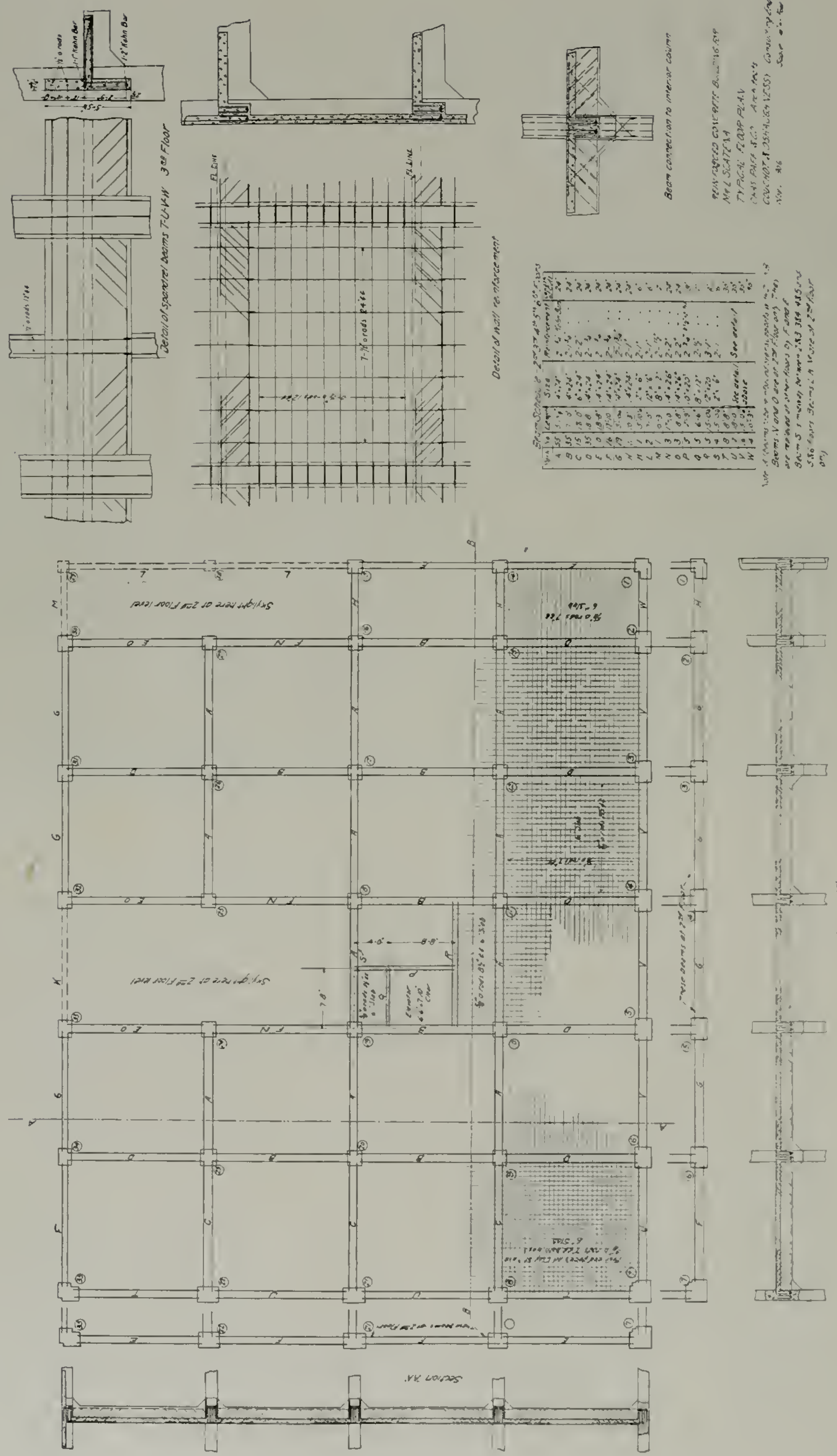
Another example, also completed, is on Sansome Street between Vallejo and Green Streets, for the Bemis Bros. Bag factory. This building is about 137′-6″ x 137′-6″ and is a credit to its builder, Mr. Lester. This is considered one of the best constructed buildings in the city. The materials were carefully selected, the labor intelligently directed, and a visit to this building will convince any prejudiced mind that reinforced concrete is a superior construction.

The Wellman Estate Building, on East Street and Drumm, Mr. Julius Krafft, architect, is also a good example of reinforced concrete construction, Messrs. Richard Keating & Sons being the constructors. This building is about

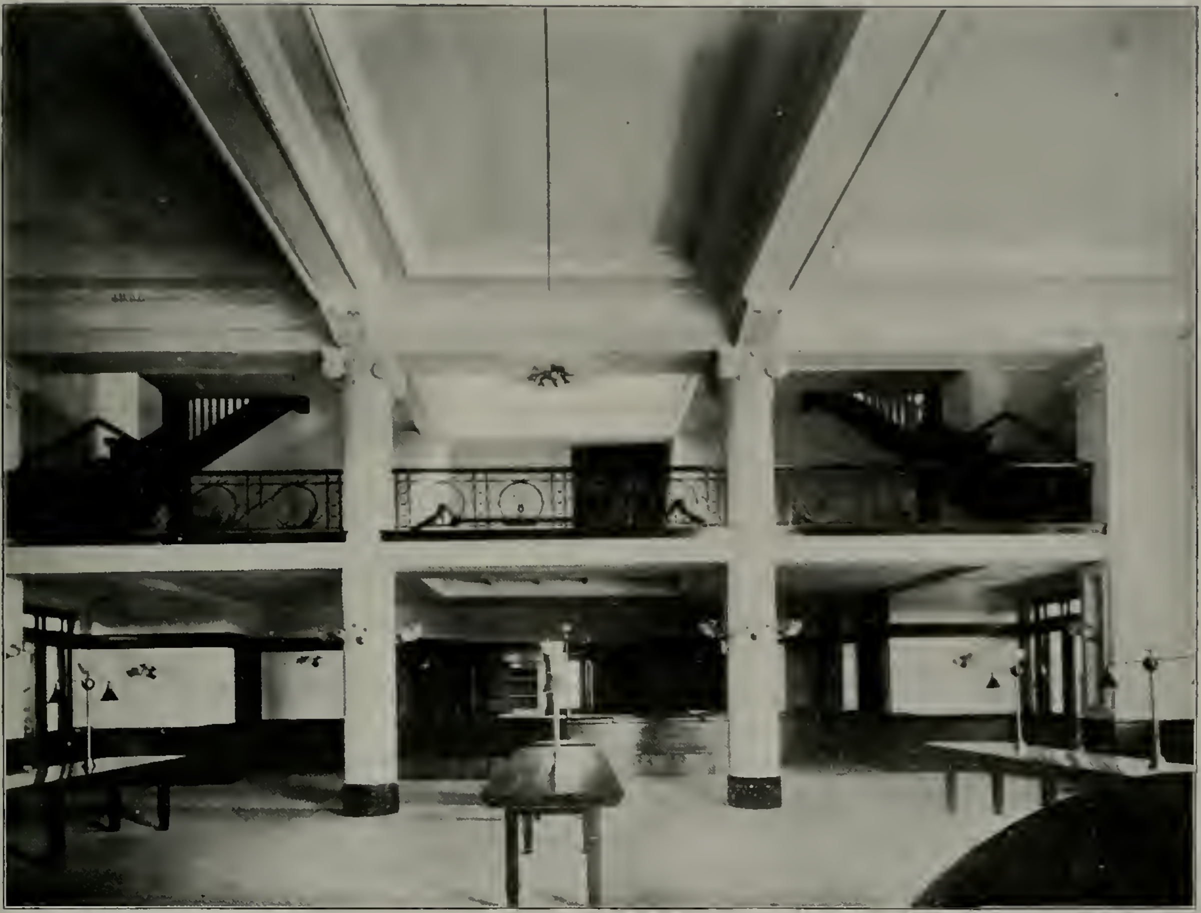


Detail of First Floor Slab

August Nordin, Architect BUILDING FOR THE GEORGE BROWN ESTATE *M. C. Couchot, Engineer*


Detail of spandrel beams T-U-V-W 3rd Floor
Detail of wall reinforcement
Beam connection to interior column
Skylight here at 2nd Floor level
Skylight here at 2nd Floor level
Section AA
Section BB

SCATENA BUILDING
Typical Floor Plan: Kahn System



MARYSVILLE PUBLIC LIBRARY

137'-6" x 160', designed for heavy loads, and for five stories, only three being built at present. The old pile foundation was intact and used in the new building.

The Scatena Building, corner of Sansome and Clay Streets, Chas. Paff & Co., architects, will be a six-story loft building designed for 150 lbs. live load, and is at present in course of construction.

The Land-Wharton Co. Building, corner of Sansome and Bush Streets, Messrs. Bliss & Faville, architects, is a seven-story loft building.

The Latham Building on Mission Street, Mr. E. Bolles, architect, a six-story loft building, is also in course of construction. Cook & Young are the builders.

All of these examples are designed according to the Kahn System of reinforcement, and in every case the construction has been in the hands of responsible and careful builders. Reinforced concrete is purely an engineering problem having its economic field just as much as steel, stone or brick, and is a very good construction when used right and in its right place. The trouble is that we have in California at the present time too few men who are conversant with it and understand its possibilities as well as its limits.

Upon the cover of this issue we present a pleasing view of the Marysville Public Library, a gift to the city of Marysville by Mr. George Packard; Mr. William Curlett, architect, Mr. M. C. Couchot, engineer. The interior construction of this building is of reinforced concrete, according to the Kahn System, and exhibits some excellent long span floor design. We present herewith also an interior view of the building, giving an idea of the proportions of the floor beams and floor panels. The main girders have a span of 32 feet, a cross section of 18" wide by 32" deep, and are reinforced by

3—1¼″ and 2—1″ Kahn bars. The main floor panels span the distances between the main girders, which latter are 17 feet center to center. The floor slabs are 11″ in depth and consist of 5″ concrete ribs spaced by 10″ tile, 12″ wide. Each 5″ concrete rib is 11″ in depth and is reinforced with 1—¾″ Kahn bar. These bars are therefore 17″ center to center.

The Artistic Treatment of Concrete *

THE consideration of concrete from an aesthetic point of view may strike the average cement user as inappropriate and impossible, for hitherto the term "concrete" at once suggested foundations, piers, dams, abutments, and now-a-days it more than likely calls to mind columns, beams, floors, walls, and, in fact, the entire structural parts of buildings. But it must be evident that our experience with this new material, new only, however, in the sense of adaptation, will show that, like all our legitimate and substantial structural materials, it, too, will prove to be susceptible of artistic treatment in design. Wood and stone architecture are as old as the hills, and the art of the mediaevalist in developing true styles with these humble materials rightfully belongs to the world wonders. Brick and terra cotta can scarcely claim quite so much distinction, although in point of artistic treatment they were brought to high development in the Gothic art of Italy. Iron and steel, however, do not fare quite so well except in a small way along the lines of purely ornamental work; for when the modern rolling mill began to turn out its products of structural shapes, and engineers discovered the wonderful possibilities of riveted sections and connections, there was a great rush for structural iron and later for steel. Everything imaginable was made of it, and more in Europe than here. Bridges and viaducts, certain classes of business blocks and public buildings, even churches and cathedrals, all vied with one another in their architecture of iron and steel, and while some notable attempts achieved a measure of success, it required many years to develop an artistic style of design. And even at this late day, structural engineers as a class seem determined to ignore the application of aesthetic principles to their designs of exposed work.

But we feel quite hopeful. There has been and is much teaching and preaching of aesthetics. Schools, periodicals, and municipal art societies are doing much to educate the popular taste and to create a demand for beauty in public works, and this campaign, whose influence is spreading rapidly, will undoubtedly bring designers to recognize and appreciate the necessity and propriety of combining beauty with utility in all visible constructions.

This problem will be greatly simplified in concrete work; for here, for the first time, we come to deal with a plastic material which can be moulded and modeled at will. Beauty, however, in structural, design is worthy of the name only when, like beauty in nature, it has character. It must not be a servile copy of the style peculiar to some other material, but, in fact, must express the individuality of its own nature and must not dissemble.

It is just this peculiarity that we must be careful of in our concrete block architecture. At present the tendency in the manufacture of these blocks is to imitate split faces of stone ashlar. This is radically wrong in principle, and should not be tolerated. A flat, smooth face will always look well. However, if a pitched or split face is desired, let it be produced by casting the block flat and then pitching off the face with chisel and hammer just as is done with stone. The clean fracture of the concrete thus exposed will be eminently effective and artistic and will have all the merit that belongs to truthfulness. Plain concrete ashlar walls might in some cases be effectively relieved by the introduction of bands of decorated blocks with some simple ornament moulded in the face, very much as is done with terra cotta, but by all means moulded rock-faced work. It is artistically bad. The frequent and constant repetition of a few regular sizes and patterns, ruins an effect which should be counted largely as accidental, but always expressive of a fine artistic sense in the selection and grouping of the individual blocks. Artificiality, imitation and misrepresentation

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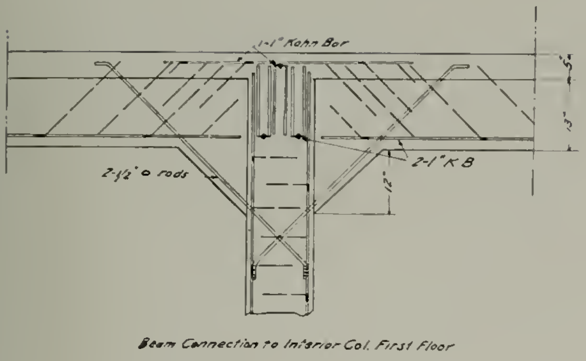
*Paper presented before the National Association of Concrete Users by Mr. O. A. Elzner, of Elzner & Anderson, Cincinnati, Ohio. Reprinted from the Engineering Record, Vol. 55, No. 2, January 12, 1907.



GEORGE BROWN ESTATE BUILDING
See Article by M. C. Couchot, Page 46

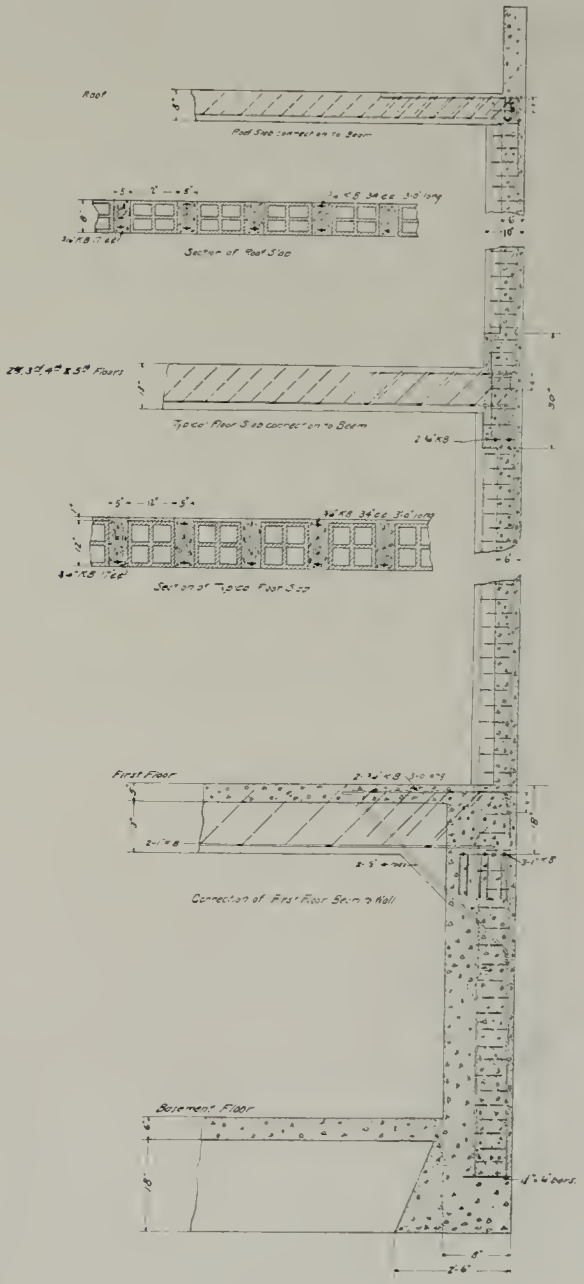
are stamped all over such work and can be recognized at first glance.

Solid concrete walls have a great advantage over the block walls in that they lend themselves much more readily to artistic treatment. This is especially true where they are used in suburban and country buildings, perhaps because of the touch of nature in the surroundings which more nearly accords and harmonizes with the broad treatment that can be so effectively employed in wall surfaces. Perhaps the best sources of inspiration that can be had for such treatment are to be found in the old Spanish Missions of California, which, although not of concrete, nevertheless at once suggest its use and above all are fine examples of the artistic value of broad wall surfaces relieved by exquisitely proportioned openings, judiciously spaced, and not infrequently embellished by a moderate use of ornamentation.

Let us say, then, speaking of domestic architecture, where walls are made of solid concrete, the surface should be as unbroken as possible, avoiding especially artificial jointing, of which such frequent use is made, obtained either by scratching a joint into the fresh mortar with which the surface is plastered, or after the removal of the forms, or by planting beveled wood strips on the inner surface of the forms, thereby molding the joint directly into the concrete.

Both methods are highly objectionable, utterly senseless and aesthetically very bad, and should be shunned. In work of moderate cost, where effects are to be sought in an inexpensive, straightforward and natural way, there can be no offense taken if the concrete is left untouched after removing the forms. In fact, this method has so much merit, that it might with perfect propriety be classed as the most thoroughly artistic. That is probably just what the builders of the old Spanish missions would have done if they had had concrete to use for their buildings. To be sure, if such treatment is to be used, some care should be exercised in the preparation of the form work, so that it will not result in the effect of a lot of patch work.

In more pretentious work several methods of treating the exposed concrete are available. A thin skin or crust of neat cement usually is found to cover the surface where concrete was deposited wet and was well tamped. This crust



GEORGE BROWN ESTATE BUILDING
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may be removed while still soft by means of a stream of water having some force, or by stiff wire brushes, in which cases the forms must be removed promptly and just as soon as the work will stand it. This, however, involves considerable danger, and should be done only by thoroughly experienced persons. If successfully accomplished, the effect of the rough surface thus produced is good and consistent, for it exhibits the material in its true nature and avoids all semblance of artificiality.

This treatment, however, entails so many difficulties that it will not be very popular, and it will be advisable to adopt some other simpler and safer method giving similar results. The surfaces can be tooled all over with a chisel, as in some classes of stone work, but while the result may be effective, it is rather expensive and slow work and will therefore be but sparingly used. It is difficult, too, to avoid loosening an occasional pebble or stone and thus spotting the surface with objectionable blemishes, and possibly opening up some internal cavities which are quite apt to occur and so starting a leak in the wall.

A simple and inexpensive, yet thoroughly practical method of securing an artistic effect, consists of covering the wall surface with a splatterdash coat of cement mortar applied by splashing it on with a paddle or a broom, or, better still, it may be first spread on with a trowel and then roughened by stippling with a stiff broom or brush or even a flat board, in which case the roughening is obtained by suction against the board. When such treatment as this is to be used it may be highly appropriate in some cases, and indeed quite interesting, to decorate parts of the surface with some simple panel work or freehand modeling. In case of panels, it is best and simplest to adopt sunken work, as this can be readily produced by merely planting a board or block of desired shape against the inside face of form work, which leaves its impress upon being removed from the concrete. Or else a reverse made of some artistic bit of carving for a panel, or over a door or window, or a frieze, etc., may be nailed against the forms, and the resulting impress will be thoroughly effective, although a much higher artistic value would be due to such work if it were modeled by hand directly in the cement mortar as it is applied and before it has had a chance to harden.

This sort of work is being done extensively and successfully in Germany where the modern style of "Nouveau Art" presents abundant opportunity for endless designs. It is already finding much favor in our country and ought to reach a high degree of development.

Moldings, especially in continuous cases, if attempted at all, should be of the simplest possible design; bold, yet of moderate projections, and free from small, delicate members. Square effects and beveled projections serve very well in the place of conventional molding and rather accentuate the character of the work and heighten its effect. Dentils of fair size can be worked in to good advantage, and with comparatively little difficulty. Such work, should, however, be used sparingly on account of the impracticability of treating the surface of the resulting small members, unless great freedom and latitude are allowable without detriment to the artistic character of the design. It is particularly difficult to do this in case the walls are to be plastered over with cement cement mortar. Where this is done, the work should be finished under the float rather than the trowel, so as to minimize the tendency to crack or craze, a great source of annoyance and disfigurement. Trowel finish, furthermore, almost invariably produces a series of blotches of different shades and textures, which, if introduced into rough work, have much artistic value, but must be classed as nothing better than blemishes in smooth troweled surfaces. Moreover, it is extremely difficult and well-nigh impossible in plastering over moldings or projecting band courses, to keep the edges straight and true as they should be in smooth finish, with the result that the poor, slovenly workmanship imparts an air of cheapness and flimsiness to the building instead of the reverse, value and substance.

Such, then, are some of the readier methods that can be employed in producing artistic effects with concrete. This humble material, so replete with possibilities, but as yet so little understood, is manifestly destined to take an important place in the construction of our buildings, and must therefore strongly influence their design. But it means long, continuous and close observance and study of its nature, its possibilities and its limitations. If designers will devote themselves sufficiently to this subject, as it so well deserves, they will discover in concrete a new and useful friend, and with its help will evolve a new architecture, full of life and character, strength and dignity, and all else that goes to make up a living style.

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EARTHQUAKE REPORT OF THE SPRING VALLEY WATER COMPANY.*

THIS most timely report will be welcomed by hydraulic and municipal engineers, in particular, but by all students of engineering, in general, because it adds another authoritative discussion to engineering literature relative to the destructive effects of the earthquake and fire upon the engineering works of the City of San Francisco. The earthquake produced a most appalling and unique situation in the City due to the breaking of the main conduits supplying the City with water, and the subsequent conflagration added equally to the seriousness of the situation. Mr. Schussler, naturally, is in a position to give the actual facts descriptive of the Spring Valley Water Company's losses and to outline most definitely the steps toward rehabilitation taken by the Water Company in those first trying days following the catastrophe.

The report, in its first three parts, or chapters, gives: 1. A description of the works of the Spring Valley Water Company as they existed last April; 2. A short historical review of San Francisco's water supply from the early 60's until the earthquake of April 18, 1906; and 3, a discussion of the earthquake of April 18, 1906, and the subsequent conflagration, especially regarding their effects on the works of the Spring Valley Water Company.

Following these three main chapters are given a number of appendices, presenting correspondence between the Company and the municipal authorities, and quoting from descriptions made by engineers and others who visited the Company's reservoirs and conduits immediately after the earthquake. The final appendix gives extracts from the Spring Valley Company's specifications for brick-work, concrete, clay embankments and wrought iron. The report closes with a large number of excellent and most valuable folding plates upon which are reproduced important maps and photographs. The plates show the extent of earthquake and fire destruction to San Francisco's water works, and give clear pictures of emergency repair problems which were quickly met and solved by the Water Company's engineers in the days immediately following the calamity.

During the past six months much has been written or said concerning the water supply problems of San Francisco in the light of earthquake and conflagration possibilities in the future. Special emphasis has been laid upon questions of fire protection and we have heard much about proposed salt water systems, new sources of supply, more city reservoirs and pumping stations, and in fact, entirely new water projects to be owned by the city. Nothing has yet been done. There is indeed much need for improvements and development in the city's water resources, but the Spring Valley Water Company is not responsible for everything. That Company has been and is most roundly criticized, but we, as citizens of San Francisco, will be wise by being reluctant to completely condemn an existing works; and we will be fair if we recognize what is good in the works of the Spring Valley Company. We should be alert to study the advantages and disadvantages of all new propositions lest we accept something good for what was better.

Mr. Schussler's report naturally voices the arguments of the Company of which he is Chief Engineer, but it is none the less valuable for that, and adds one further important contribution to the study and discussion of the water works of San Francisco from which study and discussion, let us hope, will eventually result sound and honest conclusions for the future betterment of one of the most important of the city's public works.

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*The water supply of San Francisco, California, before, during and after the earthquake of April 18, 1906; and the subsequent conflagration; by Hermann Schussler, chief engineer of the Spring Valley Water Company. Published by the company.

Whatever the faults of the Spring Valley Water Company may be, real or imaginary; whatever improvements and extensions have not been made in the past which now we appreciate keenly through the lessons of destruction, at least, it is fair and generous and right to say that the engineers of the Water Company have built soundly and safely, and their engineering structures in general are far superior to the average buildings erected in San Francisco and vicinity before, and we regret to add, also after, the earthquake.

Shear and Bond Tests of Concrete.*

DURING the past two years a number of very valuable bulletins, giving results of tests of concrete and reinforced concrete, have been published by the University of Illinois. These Bulletins are contributions from the Engineering Experiment Station of that University, of which Station Professor Arthur N. Talbot is Director.

One of the latest bulletins of the above series has just appeared, and deals with investigations inquiring into the shear and bond strength of concrete and reinforced concrete.

We recommend to our readers, who are interested in concrete work, that they carefully examine this bulletin.

Professor Talbot's conclusions are most interesting and valuable, and we append them in full.

CONCLUSIONS FROM SHEAR TESTS.

"1. It is difficult to devise a form of test specimen and a method of testing which will satisfactorily determine the resistance of concrete to shear. The difficulties lie in the inability to secure an even distribution of the shear over the shear action, in the high cutting and bearing stresses developed, and in the complications formed by the compressive, tensile, and bulging and bursting stresses developed. The forms of test specimen here used are not fully satisfactory, but information concerning the shearing resistance of concrete may be drawn from the tests as a whole, and tentative values selected. A test specimen in the form of a beam and in which the load is applied evenly over the depth of the beam instead of on the top is suggested."

"2. The resistance of concrete to shear is dependent upon the strength of the stone as well as upon the strength of the mortar, and for the richer mixtures the strength of the stone probably exercises the greater influence. With hard limestone and 1–3–6 concrete 60 days old the shearing strength may be expected to reach 1,100 lbs. per square inch, and with the 1–2–4 mixture 1,300 lbs. per square inch. It seems very probable that the resistance to simple shear is considerably higher than this, and that tests made with the load applied evenly over the shearing action will verify this."

"3. Since the compressive strength of concrete is influenced largely by the strength of the cement and the shearing strength is much more influenced by the strength of the aggregate, it does not seem proper to express the shearing strength in terms of the compressive strength. However, this is frequently done and is of advantage in gaining a conception of their relative action. It appears that the shearing strength is, in general, at least 50 per cent of the compressive strength, and that it may exceed 75 per cent. The apparent exception to this is explained by the high values obtained in the 1906 compression tests. These conclusions agree in a general way with the statement of Feret and others that the shearing strength is as much as two-thirds of the compressive strength. Evidently the shearing strength of concrete is several times its tensile strength."

CONCLUSIONS FROM BOND TESTS.

"1. Little difference is found in the bond resistance per square inch of surface of bar in contact with the concrete whether the bar is imbedded 6 inches or 12 inches. Evidently a length may be found beyond which the stretch of the steel would cause uneven distribution of the bond stress along the length of the bar and cause failure to begin at the point of greatest stress in the steel and thus give results not representative of the real bond resistance. This limitation applies to length for use in experimental tests of bond. In simple beams the bond stresses are applied along the length of the bar, and stretch and bond exist together.

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*Tests of concrete: 1—Shear: 2 Bond: by Arthur N. Talbot. Bulletin No. 8 of the University of Illinois Engineering Experiment Station.

"2. The richer mixture of concrete gives somewhat higher bond resistance than the learner—the values for the 1–2–4 concrete averaging, say 10 per cent to 15 per cent higher than the 1–3–5½ concrete. For plain round mill steel rods the average for the bond resistance ranges from 350 to 450 lbs. per sq. in. of contact surface."

"3. The flat bars give much lower resistance than round bars. Only three tests were made with flat bars, and these may not be representative. It may be noted that the results with flat bars are much lower than tests made elsewhere. It should also be noted that for a bond stress of 125 lbs. per sq. in., the tensile stress developed in the bar was only 9,000 lbs. per sq. in."

"4. The value of bond resistance will depend upon the smoothness of the surface of the bar, the uniformity of its diameter and section, the adhesive strength of the concrete, and the shrinkage grip developed in setting. The effect of smoothness of surface and uniformity of diameter and section is seen in the tests made with cold rolled shafting and tool steel. The average bond developed with cold rolled shafting and tool steel was 147 lbs. per sq. in. of contact surface as compared with about 400 lbs. per sq. in. for ordinary plain, round, mild steel rods. It should be stated that not only was there a very noticeable difference in the smoothness and finish of the surface of the rods, but the section of the cold rolled shafting and tool steel was very uniform, the diameter not varying more than 0.0001 or 0.0002 in. at ¼ in. intervals throughout the length, while mild steel rods will vary as much as 0.0015 in. It is to be expected that the smoothness and uniformity of section of drawn steel wire will operate to give low values of bond resistance, though, of course, as the section of wire is small compared with the circumference, the bond stresses developed when wire is used are relatively small. Attention is called to the fact that in the reinforced concrete beams described in Bulletin No. 4 the bond streses developed in beams failing by tension of the steel, diagonal tension of the concrete or other similar methods amounted to from 73 to 193 lbs. per sq. in.

Table 20 Summary of Shear Tests

FORM OF SPECIMEN	YEAR	KIND OF CONCRETE	METHOD OF STORING	NUMBER OF TESTS	STRENGTH LB. PER SQ. IN.			RATIO OF SHEAR TO COMPRESSION	
					SHEAR	COMPRESSION		CUBE	CYLINDER
						CUBE	CYLINDER		
Plain plate.	1905	1–3–6	Air	9	679	1230	....	.55	....
	1905	1–3–6	Water	7	729	1230	....	.59	....
	1906	1–3–6	Damp sand	4	905	2428	1322	.37	.68
	1906	1–3–6	Damp sand	1	968	1721	1160	.56	.83
	1906	1–2–4	Damp sand	5	1193	3210	2430	.37	.49
Recessed block ...	1905	1–3–6	Air	17	796	1230	....	.65	....
	1905	1–3–6	Water	6	692*	1230	....	.56	....
	1905	1–3–6	Water	5	879	1230	....	.71	....
	1906	1–3–6	Damp sand	4	1141	2428	1322	.47	.86
	1906	1–3–6	Damp sand	1	910	1721	1160	.53	.78
	1906	1–2–4	Damp sand	5	1257	3210	2430	.39	.52
Reinforced recessed block.	1905	1–3–6	Air	4	1051	1230	....	.86	....
	1906	1–3–6	Damp sand	4	2181	2428	1322	.75	1.38
	1906	1–3–6	Damp sand	1	1555	1721	1160	.90	1.39
	1906	1–2–4	Damp sand	5	2145	3210	2430	.67	.88
Restrained beam...	1906	1–3–6	Damp sand	4	1313	2428	1322	.54	1.00
	1906	1–3–6	Damp sand	1	1020	1721	1160	.59	.88
	1906	1–2–4	Damp sand	6	1418	3210	2430	.44	.58

Even at the breaking load, then, the bond stress developed in the mild steel rods was far below the bond resistance found in these tests."

"5. In these tests the bars began to slip when the maximum load was reached. After slipping began, the resistance to motion was still considerable. This running friction, taken when the bar had moved about $\frac{1}{4}$ in., amounted to 54 per cent to 72 per cent of the bond developed in the case of mild steel bars and to 32 per cent to 49 per cent in the case of the cold rolled shafting."

Roman Stone

IN 1899, Chas. W. Stevens of Chicago, Ill., patented a process for making concrete stone by the wet process by casting a properly mixed aggregate in an absorbent mould by the use of which the surplus water from the mixture is absorbed and held by the surrounding mould, which serves as a wet covering to protect the cast and supply the proper amount of water while the cement is setting, thus making the proper supply of water automatic.

This invention at once gave a process by which a product could be made that was in fact a stone of the highest order and has revolutionized the art of making stone.

Not only does the process produce an article with stonelike texture, dense and homogeneous throughout, but one whose color is found to be as light and more uniform than that of most building stones and entirely free from the objectionable features of manufactured stone made by other processes.

Every precaution known to the profession is taken to produce perfect results. Materials are all tested before being used. The stone used is carefully selected building stone. Artificial coloring matter is never used, but by selection of stones from different quarries, it is possible to obtain different shades running from white to buff.

The crushed stone, before being mixed with the cement, is double-screened and separated into different sizes which are reunited in the exact proportion to properly fill all voids. It is automatically gauged with a proper amount of water kept at a constant temperature. The casts, after being made, are all properly seasoned before shipping.

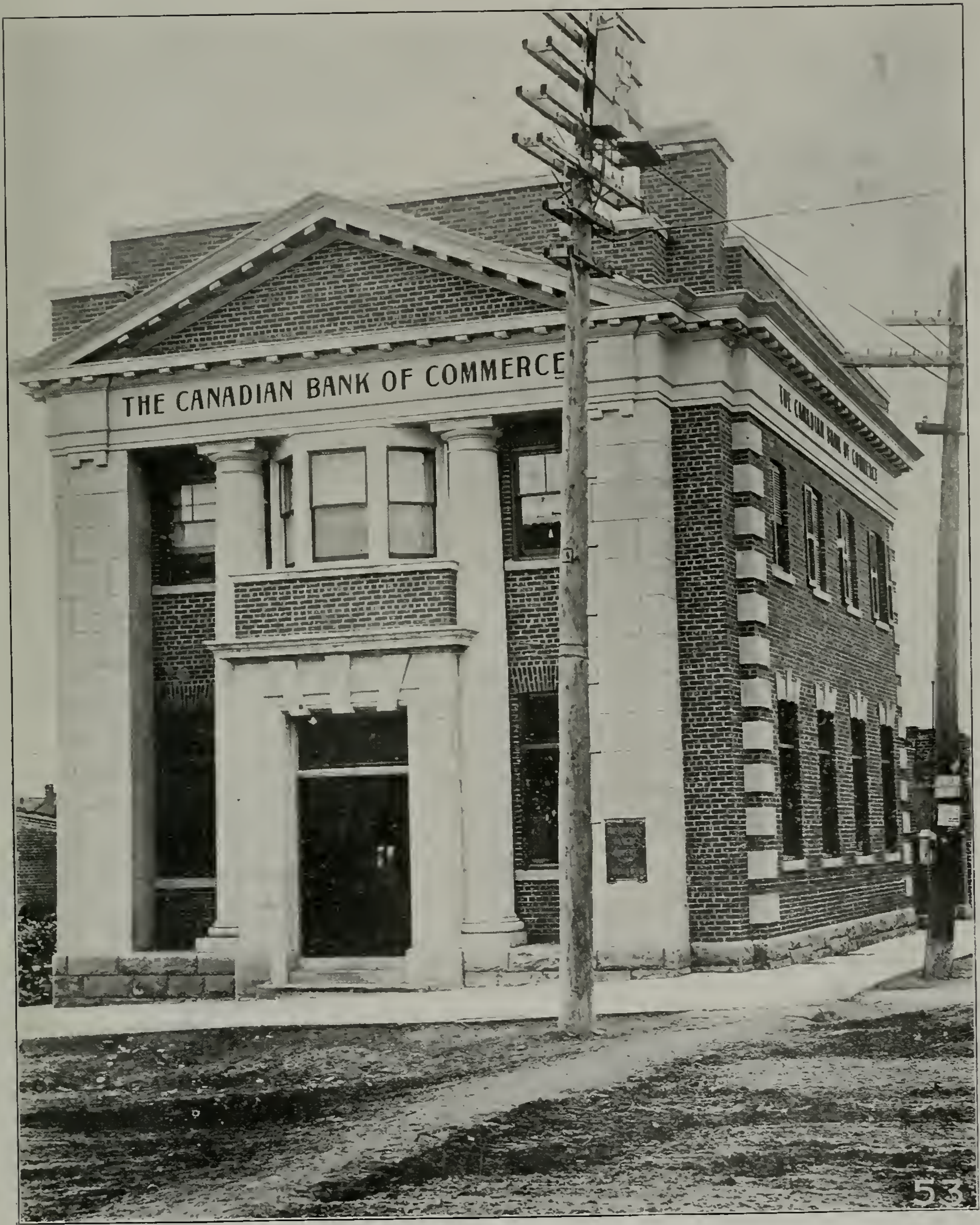
On account of the perfect condition for crystallization produced by the Stevens process, Roman Stone utilizes all the strength of the cement. In the large Toronto fire of 1904, in the John D. Ivey Company's warehouse, it stood the test without injury. A block of Roman Stone can be heated to redness and thrown into cold water without cracking.

The April fire in San Francisco demonstrated that granite splinters quickly when subjected to heat, and those who have passed the Fairmont on California street and witnessed the breaking of the hugh blocks of granite which came from the old wall have wondered at the ease with which it was done. A slight fire is built under the stone until it is heated moderately when a few buckets of water thrown on crack the blocks into many pieces.

An interesting experiment was carried on at the Roman Stone factory at Cornwall, California; granite was taken from the Rocklin Granite Company's quarry in Placer County, crushed and made into stone in the proportion of one to four. After being throughly cured, it was placed in the fire of a blacksmith forge and the blast continued until the stone was at a white heat on all six sides (a condition never met with in ordinary fires), the block was then withdrawn and thrown into water; examination showed that the stone had not cracked nor disintegrated and, when broken by hammer the following day, appeared to have gained in strength.

For ease in handling in the factory and on buildings, double hooks are set in the back and top beds of all large stones, and when the stone is to be used as a facing for reinforced concrete buildings, these hooks provide an opportunity to use the stone as one side of the form. It further allows a steel hook to be placed into the hook already in the stone while the other end may be imbedded in the concrete wall, thus making the facing of the building an integral part of the structure.

Roman Stone is dense; it weighs as much as natural stone and 50 per cent more than stone manufactured by the old and antiquated dry tamped method. The compressive strength of



Darling and Pearson, Architects THE CANADIAN BANK OF COMMERCE

Ornamented with "Roman Stone", executed by the Roman Stone Company of Toronto, Ontario.

Darling and Pearson, Architects. ENTRANCE TO TRINITY COLLEGE, TORONTO
Building of Roman Stone.

Roman Stone at one month averages 2,500 pounds per square inch, and constantly increases for years.

The weathering qualities of Roman Stone so commended themselves after several years of exposure in Toronto that it was selected by Messrs. Green & Wicks, architects, for facing the Gate house and Screen house for the Ontario Power Company at Niagara Falls.

The greatest virtue of Roman Stone is its light, uniform color. It is perfectly homogeneous throughout; it can be cut and carved like natural stone, and resembles it so closely that experts are deceived. It is the only manufactured stone which has a texture allowing it to be hand-carved.

Roman Stone absorbs less moisture than natural sand or lime stone, and less than one-half of what ordinary dry-tamped stone absorbs.

Roman Stone can be reinforced with steel, thus making it capable of sustaining the heaviest loads.

The cost depends entirely upon the design. In nearly every case it is below natural cut stone, and under some favorable conditions it is nearly 50 per cent less.

Many tests have been made showing the strength and absorption of Roman Stone, the results of which vary somewhat with the age of the material. The results obtained indicate that it has an average compressive strength of 2,500 to 5,000 lbs. per square inch, and that its absorption varies from 2½ to 5 per cent, depending upon the nature of the stone used and the age of the test samples.

Competent authorities will advise that tests to be of any value should be made with samples at least six months, and preferably one year of age.

Comparative tests at less age are useless, because in the process of casting plastic concrete, strength is developed slowly, but increases uniformly with years, while casts made with a small amount of water develop their full strength in a few weeks, after which they gradually deteriorate.

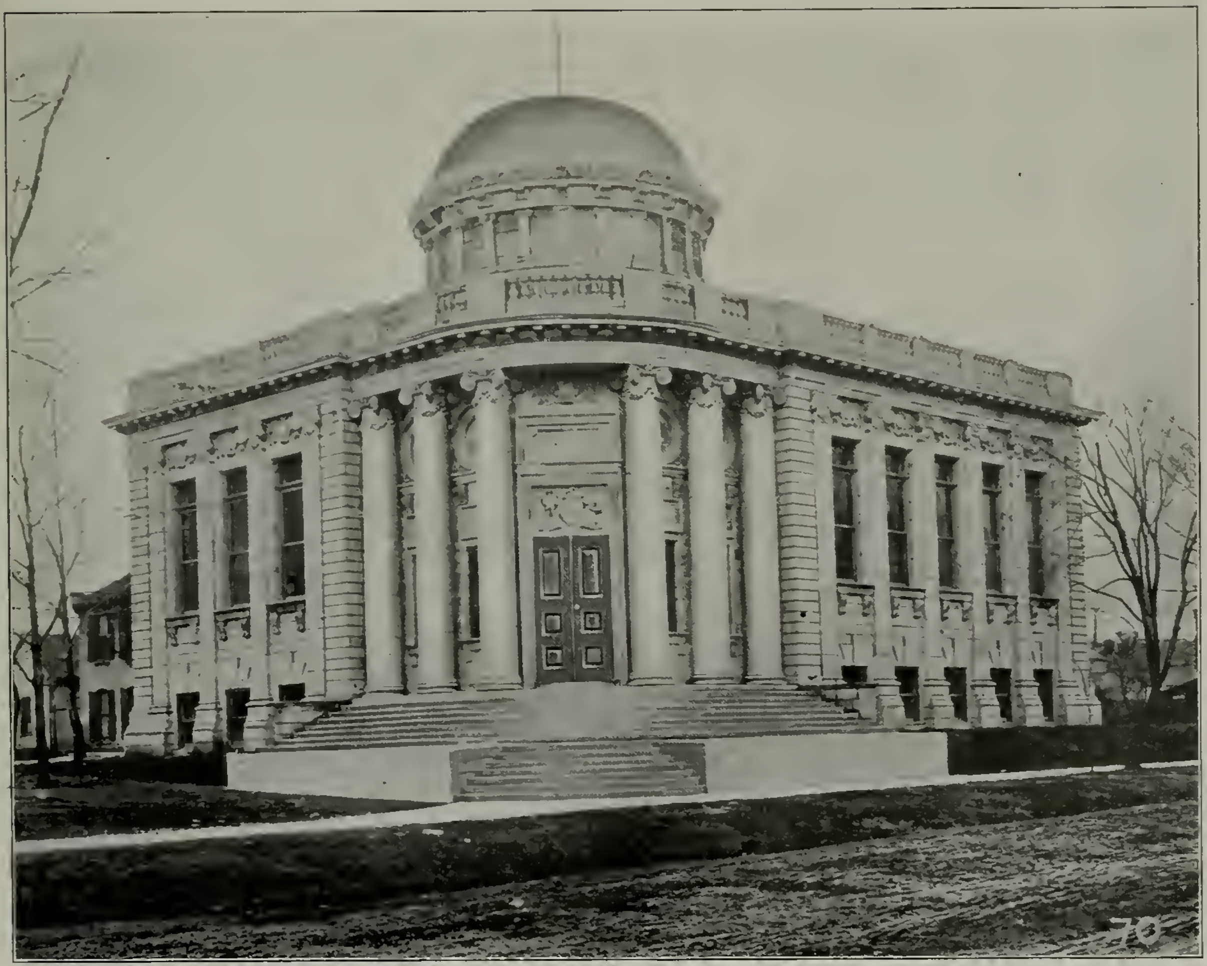
This characteristic of wet mixed concrete is one of its greatest virtues.

The following tests, made by E. S. Larned, Member Am. Soc. C. E., will demonstrate the effect of water on mortar, gauged with different percentages of water by weight. The figures show the tensile strength in pounds per square inch:

7 days	261	392	301
28 "	344	447	407
3 months	344	494	430
6 "	392	536	454
12 "	300	478	474

These results show that in using less than 14 per cent of water the strength is fully developed at six months, after which it begins to deteriorate, while the mixtures made with a greater amount of water develop their strength slowly but uniformly without deterioration.

Charles A. Matcham, one of America's greatest concrete experts, states that tests made at the age of one year or more have fully demonstrated that the loss of strength in its early stages, due to making the concrete plastic, is more than regained in time.



A. Piche, Architect

CITY HALL, STE. CUNEGONE

A Reinforced Concrete Building faced with Roman Stone.

Cinder Concrete; Its Performances in Reinforced Concrete Floors in San Francisco Buildings*

PART A.

SAN FRANCISCO, October 18, 1906.

To the Executive Committee,
Structural Association of San Francisco.

GENTLEMEN:

AT your request we have made an examination of the cases of corrosion of metal in cinder concrete floors, brought to your notice by Mr. C. F. Wieland.

We have examined the specimens of floor slabs at Mr. Wieland's office and later each of us carefully examined the floors in the following buildings:

Sloane Building, Post Street, age 8 years; all floors cinder concrete and expanded metal on I beams about 6 feet apart.

Crocker Estate Building, First and Mission Streets; floors the same as in the Sloane Building; built in 1902.

Rialto Builidng, New Montgomery and Mission Streets; lower floor the same as the above; upper floors of rock concrete and Johnson bars; built in 1902.

Wells Fargo Building, Second and Mission Streets; cinder concrete and galvanized wire fabric. We found the fabric blackened upon the surface, but the metal was not corroded.

The cinder concrete is somewhat porous with occasional voids and also contains coal from dust up to lumps ¾-inch in diameter; rust spots occur in the concrete and where such spots are in contact with the metal the corrosion is severe. The rust spots are sometimes 1 inch across, quite soft and easily removed by the finger nail. Occasional splinters of wood occur in the concrete, which shows that the heat was not severe, as the wood is not charred. From the position of the floors it is certain that no water has reached the concrete since April 18th, and that the corrosion was prior to the fire; but the corrosion appears to be more marked where floors have been exposed to the rains since the fire. The corrosion is irregular in amount; in some cases the expanded metal is only slightly rusted, while in others it is entirely destroyed. Several places were noticed where a small semi-circular patch had been removed from the edge of a metal strip, also at times it crossed the surface of the strip in a line, which suggests it followed a surface crack in the metal. There seemed to be a tendency to corrosion at certain points in the diamond mesh, which would indicate that the metal had been strained in the process of slitting and expanding, but there is not positive proof of this.

The extent of the corrosion is great enough to seriously endanger the safety of the floors and it is not probable that the floors would have supported their loads longer than one to three years.

CAUSES OF CORROSION

Various reasons for the corrosion of metal in cinder concrete have been given. Quite prominent among them is the belief that it is due to sulphur in the cinders. In order to corrode metal, the sulphur must first oxidize to $H2\ SO4$ and if there is any notable amount of cement present it would be absorbed by the cement. It is not probable that sulphur can rust metal unless it is either close to or it the concrete is very deficient in cement.

Prof. Norton (see Report III, Ins. Exp. Sta., Boston), made several hundred experiments and it is his opinion that the corrosion is not due to sulphur but to a too dry mixture, which fails to properly coat the metal, and air and moisture enter the pores of the concrete and set up rusting action. This view is also confirmed by Booth Garrett & Blair, who state that a cinder containing two-tenths per cent of sulphur will not corrode steel, provided the concrete had sufficient cement mortar to close up the voids and cover the metal.

It is stated by makers of sheet metal pipe that there is a marked difference in the life of a pipe that comes here, rail or sail. The exposure to a sea voyage renders the usual protection by dipping useless, as the surface rusts under the dip and the coating falls off the pipe. The pipe-makers state that this happens

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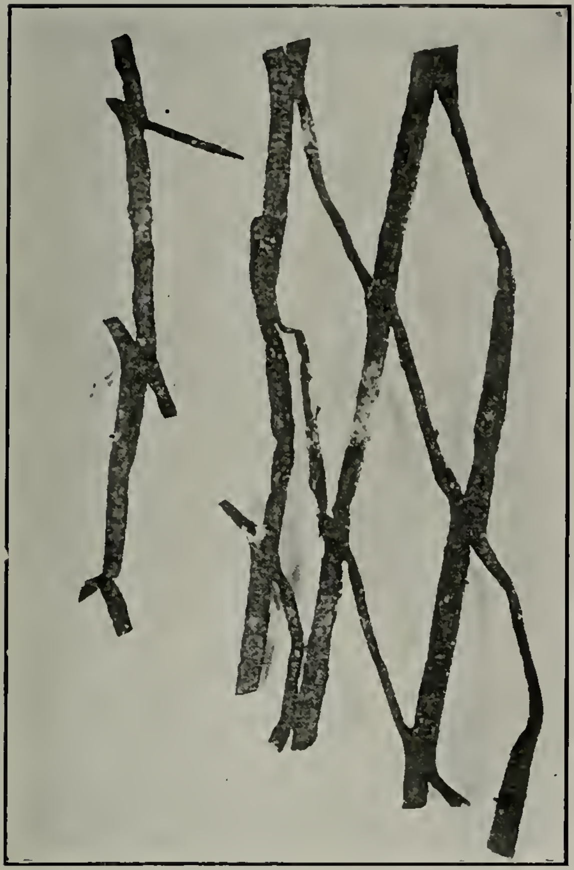
*Presented as a final report to the Structural Association of San Francisco, meeting of October 18, 1906, by Messrs. C. F. Wieland, L: Wagoner and T. H. Skinner, Special Sub-Committee; Presented in parts.



Expanded Metal From the Rialto Building

to sheets apparently unaffected by rust when dipped. The slitting and expanding of the metal sheets may cause minute cracks upon the surface of the metal. If this be combined as in the case here with a thin slab of concrete, porous and ranging in thickness from 2¾ to 4½ inches, with the metal in many cases flush with the bottom of the slab, there is an ideal condition for the rusting of the metal. Hambuechen (Univ. of Wis.), made a study of corrosion and has proved that a point of segregated carbon is always an initial point of corrosion and has shown that burnishing steel or iron protects it notably by removing the focal points; he has proved that rusting sets up a feeble current, the segregated carbon points acting as poles. Possibly the presence of much free coal and coke in the cinder may be a contributing cause by setting up feeble electric currents or by conveying them from earth currents due to leaks from trolley wires. In the Rialto Building the upper floors are of sound rock concrete and where the Johnson bars are exposed they are free of rust and appear to be in the same condition as when placed there.

The Bullock and Jones Building, a ten-story building, had all floors of rock concrete and expanded metal. During the repair of the steel frame,it was necessary to remove quite a number of floor panels and the metal was found in as good condition as when placed. The foreman who removed the floors told us that he had only observed two slightly rusted places, which he thought was prior to its being used in the floor.

PREVENTION.

Professor Norton advises coating the metal with a paint of neat cement or dipping the metal in a thin grout. This is hardly a practicable method here and it is doubtful if it could be done with that thoroughness that such work demands.

RECOMMENDATIONS.

That the Structural Association of San Francisco try to amend the present building law so as to exclude the use of cinder concrete in floor slabs for fireproofing.

That provision be made in the building law for the examination and tests of any existing cinder concrete now in use or that may be used at some later period.

(Signed) Luther Wagoner,
Civil Engineer.
Theodore H. Skinner,
Architect.

Part B

ADDITIONAL NOTES ON CINDER CONCRETE

BY L. WAGONER.

Mr. R. Keatinge, contractor for the Wells Fargo Building, says that the cinder concrete contained small reddish brown rusty spots, about pea size. About two months after the floor slabs were laid, the rust spots probably swelled and forced off a cone of concrete about ¾-inch deep and four inches in diameter. No satisfactory explanation was found for this at the time. He also says the cinders were not clean.

Mr. Thomas W. Brooks, of the National Tube Company, San Francisco, quotes a number of cases of corrosion where pipes, both cast iron and steel, laid in cinder filled streets in Pennsylvania, were destroyed in a short time. He says that this was so marked in the case of natural gas lines that they required a clay filling of at least twelve inches to surround the pipe, where laid in cinder fills, to protect the pipes from pitting. From a book compiled by the National Tube Company for their private use, a number of similar cases were noted.

All of which seems to indicate free sulphuric acid in a very dilute form as the destructive cause.

The swelling of the pea-shaped rust spots above noted, is probably due to additional oxidation with increase of volume. It is stated that iron produces ten times its volume of red rust, which would perhaps explain the forcing off of the cones of concrete mentioned by Mr. Keatinge.

Part C.

REPORT BY C. F. WIELAND

San Francisco, October 18, 1906.

To the Executive Committee,
Structural Association of San Francisco,

Gentlemen:—

I append the following to the report of Messrs. Luther Wagoner, C. E. and Mr. Theodore H. Skinner, Architect, on the subject of "Corrosion of Metal in Existing Cinder Concrete Floors."

It has long been known that corrosion of iron and steel will result from its contact with coal or cinder. In response to my inquiry, I have the following letter from Mr. W. H. Nesbit, National Steamship Co., Monadnock Building, San Francisco.

"Replying to yours of the 4th instant, I advise that in our practice (Thomas Cronin Co., Pittsburg), where cast or wrought iron pipe lines are run through cinder or ash piles, or roads constructed of either material, we completely surround the pipes with gravel or rock concrete. The armor of concrete varies in thickness from 4 to 12 inches, dependent upon whether the excavation is through cinder entirely or earth mixed with cinder.

"The reason why gravel or rock concrete should always be placed around pipes placed in cinder excavation is that, unprotected, the acids eat through the metal. In one case—that of a borough in which we laid the entire water system, the distributing main ran through cinder for several squares. At the time we suggested to the Water Company that the pipe should be laid in concrete, but were told that so doing would entail an unnecessary expense. The result was that in eight months time, we were asked to replace that part of the system which had been laid direct in cinder, although the original pipe had been heavily coated with tar paint. This circumstance was due entirely to the action of the impurities in the cinder, for there were no underground wires, nor street cars of any description, in the place."

In volume 23, 1902, Transactions of American Society of Mechanical Engineers, is a description of a "Steel and Concrete Coal Storage Plant," by Franklin M. Bowman, Structural Engineer, Riter-Conley Co., Pittsburg. Six and seven thereof, Mr. Bowman states:

"Section 8; It is important that the inside of the storage be entirely lined with concrete, so that no part of the supporting steel work is exposed to the corroding action of the coal."

"Public attention has recently been called by Mr. Sooysmith to the possible danger due to corrosion in tall steel frame office buildings, but danger from this source is largely accentuated in a cold storage, with sulphur and other corrosive substances. For this reason, where a permanent and costly structure is to be built, as is usual in the case of large power houses, coal bins lined with steel plates should not be used, as they are liable in time to become a menace to life and property."

"Section 7; In the case of Lowell, all main columns, and as far as possible the intermediate columns, are entirely covered with concrete; again, the essential point is to have the steel framework, and especially the joints, entirely surrounded and imbedded in concrete, as under these circumstances the steel will not corrode."

The use of cinder aggregate, in coal burning districts, encourages itself by the fact that it can usually be obtained free of cost. Then too, its advocates contend that the lightness of a cinder concrete slab makes it possible to design a supporting structure of lighter and more economical members. Common sense should forbid the use of a substance which can be crushed to powder under foot, and makers of reinforcement should eliminate from their trade literature all reference to cinder concrete and thus free themselves from all moral responsibility which is implied in giving data involving its use. The relative strength of cinder concrete slabs to rock concrete floor slabs of equal thickness is as one is to two, a fact which is also in its disfavor.

If cinders were invariably "clean," free of coal and coke, and then incorporated in a dense mixture; their use as a fireproofing only might be sanctioned. The endeavor has been constant on the part of steam users to discover a device which will completely free, in a commercial manner, the heat units contained in coal, as witness the many automatic stokers and the Hydro-Carbon Furnace or Burner, and although these devices give excellent service, they do not always free 100 per cent of the fuel.

Therefore, even were coal and coke possessed of strength, neither should be incorporated, in any degree, in a covering intended to be a fireproofing; for, as stated, it is well known that heat values are contained in the cinder, or among the cinder, as drawn from the ash pits.

That the poultry netting in use in the Wells-Fargo Building is not more visibly corroded, is doubtless due to the fact that in the process by which wire is successfully drawn to its final gauge from the rod, a skin is given it, which ensures it a longer life than rolled sheet metal subsequently sheared. Some specimens of the netting in question which I have in my possession show corrosion. Rolled sheets may have the same physical and chemical characteristics as possessed by the rod out of which the wire for poultry netting is drawn, but the subsequent shearing surely lays bare the "vitals" of the sheet to more ready attack by the elements of corrosion than would be the case with the frequently drawn wire. I contend against the use of any iron or steel in contact with cinder concrete; and I believe it no easy task to find cinder free of heat units and corrosive elements.

Mr. Keatinge having mentioned the blowing which took place in the cinder concrete in the Wells Fargo Building, it is interesting to cite the experience of like character touched on by Mr. C. Newton Russell, Borough Electrical Engineer, Metropolitan Borough of Shoreditch, London, in his paper on "Refuse Destruction by Burning, and the Utilization of Heat Generated," see Transactions Amer. Soc. Mech. Engineers, vol. 25, 1904. In the refuse which finds its way out of domestic kitchens, iron and steel in the shape of pins, small screws, tinned plate scraps, etc., is of course met with, and in the sorting such as done in some plants for the destruction of garbage, cannot be recovered. In paragraph 28 of Mr. Newton's paper, he speaks of the use of the clinker, resulting from the refuse destruction, in the making of roads and sidewalk slabs, but without indicating that metal reinforcement was employed by Mr. E. J. Lovegrove, Borough Engineer to the Horsey Borough Council, London, in the slabs. Mr. Newton states:

"Mr. Lovegrove had also used the material for plastering and rendering to a somewhat large extent; but while it made a very hard plaster, there was one defect which showed itself to a greater or less extent, namely: That after the plastering or rendering had been finished for a few months, signs of blowing had occurred, causing small circular flaws about $\frac{1}{4}$ to $\frac{1}{2}$ inch in diameter in the face of the plaster; on examination these appeared to be due to minute particles of iron in the clinker. In the case where this occurred, the small pieces were cut out and made good, and no further trouble was experienced."

I feel that the purpose of the investigation into "Corrosion of Metals in Existing Cinder Concrete Floors" will have been served when such reinforced floors in all parts of the land have been cut into and inspected, and the use of cinder concrete in connection with any kind of steel and iron has been prohibited in all building ordinances.

(Signed) C. F. WIELAND, C. E.

DISCUSSION BY LETTER—CINDER CONCRETE:

San Francisco, September 22, 1906

John B. Leonard, C. E.,
Chairman, Committee on Reinforced Concrete.

Dear Mr. Leonard:—

I am off for Boston to-morrow, so shall be unable to attend the next committee meeting.

Please report that I examined the stone concrete in the Wells-Fargo Building, reinforced with three different types of reinforcement in the same slab; 1st, Ransom bars; 2d, ordinary galvanized iron chicken fencing; 3d, galvanized iron ribbon grills, and found no rust of consequence on any of the three. The concrete was a dense mixture. The spans were about 8 ft., cored out "see figure A," to a thickness of $2\frac{1}{2}$ inches, leaving ribs about 1 foot on centers, with a Ransom bar in each. The poultry net runs under the bars and follows closely the contour of the soffit of the slab; the ribbon mesh is placed near the top surface. I should think the construction must have been expensive, for, in addition to the above, there were Roebling suspended ceilings everywhere, but they did protect, and no damage was done by fire to the floor slabs.

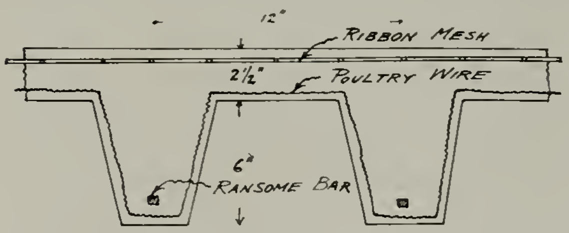


FIGURE A

Yours truly,
(Signed) THEODORE H. SKINNER.

Notes on Construction and Erection of Steel Work for High Steel Framed Buildings

By Oscar Daniels

MY subject, construction and erection of buildings, is a very wide one, and covers a broad field; for instance, it embraces the advisability of steel buildings, the design, the connection, the erection and the painting. Each of these subjects if treated properly would constitute a paper in itself, therefore, I intend only to mention a few points under each subject as they have come to me from experience.

THE FIRST SUBJECT—STEEL BUILDINGS.

In my opinion, it is not necessary to argue the advisability of steel buildings, as they have come to stay and are now accepted as the best and safest. I am not forgetting concrete constructions. Concrete may be safe for a certain class of buildings, but, as yet, it has not been accepted as a safe construction for high buildings. Concrete is never considered better than steel, and is only used for two reasons; Cheapness and time. Of course, being in the steel business, I may be prejudiced, but in my honest opinion, I can sincerely say that the new San Francisco should be constructed of steel wherever possible, and all steel protected by concrete. The earthquake and fire has proven this conclusively.

THE DESIGN.

The design of a steel building is a technical subject, one that should be treated by an engineer; therefore, I intend only to make a few suggestions, using as a basis the buildings I have erected, and my general experience. Of course, engineers, as all professional men differ. The differences involve weight, bracing, connection, riveting, bolting and painting. I have erected buildings in different parts of the country, similar in kind and to be used for the same purpose; but different in design, the cost of erecting in one case being much greater. As these designs were made by engineers of high reputation and standing, it does not become me to criticize; nevertheless, I have formed some opinions on the subject, which can be taken for what they are worth.

1. I do not believe in light constructions. A building may be designed light but safe, nevertheless, such a building is not perfectly rigid, and is easily thrown out of plumb by any unusual strain; whereas, if the construction were heavier, it would retain its position. Experience has taught us that we must design buildings in this city to meet unusual strains if necessary, and a few thousand dollars for additional cost of steel should deter no one, as everything in the building is supported by the steel.

2. The question of bracing is one that has held the attention of many prominent engineers, and has been the subject of many papers. In designing how a building should be braced, the principal thing to consider is the area, height and exposure. The smaller the area, the greater the wind bracing. The most definite types of wind bracing employed are sway rods, knee braces, lattice and plate girders. In my opinion, plate girders, ranging from thirty-six to sixty inches, are the best and most satisfactory wind braces.

CONNECTION.

The first steel buildings erected with two story columns were designed with columns staggered as to splices. This means columns breaking joints in different floors. This, of course, made the erection of steel more expensive. In the accepted designs of today, with few exceptions, the columns splices break even. They claim this does not in any way weaken the structure, but does reduce the cost of erection. I express no opinion on this question, but can only say I have erected a great many large buildings where the columns splices break even.

ERECTION.

In skeleton buildings, present demands as to rapid construction make the method of handling the steel of great importance. The demands of owners and architects for rapid completion of contracts serve to make the erection scarcely less important than a good design. Many different systems have been tried for erecting the steel frame, but the methods employed vary so much, that no general practice can be classed as standard. For the successful erection of a steel frame, much depends upon an accurate placing of the column bases. Where cast iron base plates are employed, these should be carefully set and tested both as to position and level. If the columns rest directly on the grillage beams it is more difficult and consequently more expensive to set them exactly in their proper position. I have erected buildings where both methods were employed, and have found the cast bases the most satisfactory in every respect. I have employed on different buildings, guy derricks, travellers of different designs, and shear leg boom derricks. A traveller will be found advantageous on a building up to eight stories high, when the floor plan is of such dimensions that a traveller may move from end to end of the building. For instance, I employed travelers to very good advantage in erecting a warehouse in Pittsburg, covering an area of four hundred by four hundred and eighty feet, six stories high, consisting of nine thousand tons of steel. However, I find, from experience, that the shear leg boom derricks are the most satisfactory for the erection of all high buildings. I recently erected in Chicago, an eighteen story office building, consisting of four thousand tons of steel, in seven weeks (eight hour day) employing three shear leg boom derricks.

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*A paper read before the Structural Association of San Francisco, Meeting of October 18, 1906.

RIVETING AND BOLTING.

It is the general opinion of engineers and is accepted as a fact, that the best connections are riveted ones. The only question for me to consider is whether it is necessary to rivet all connections. The expense of riveting is one of the main costs of erection. I believe that all kinds of building connections to columns should be riveted, but I also believe that in some cases, intermediate beams should be bolted. For instance, a high building on a small area should be entirely riveted, whereas one of large dimensions would only require the column connections riveted. This is now being done a great deal, the only person profiting by it being the owner. The designer of a building can, by careful study, in a great many cases, reduce the amount of field riveting and not weaken the structure, and in this way save money for his client.

PAINTING.

Steel to be encased in concrete should not be painted at all. When I was erecting the steel for the new Chicago Post Office, the following test was made: A beam was painted and encased in concrete, another beam was encased in concrete without being painted. The beams were left for a considerable time, and then examined. The concrete on the painted beam was removed in large sections, showing that the concrete adhered only to the paint, whereas the concrete on the unpainted beam was stuck firmly to the steel itself and had to be chiseled off. Where the steel in not to be encased in concrete, it should be cleaned of all shop scale and given one coat of boiled linseed oil, and painted before and after erection.

SOCIETY MEETINGS

TECHNICAL SOCIETY OF THE PACIFIC COAST

I. Meeting of January 4, 1907

THE meeting was called to order by the Vice-President, Mr. Franklin Riffle, in the rooms of the Mechanics' Institute, San Francisco.

Mr. Marsden Manson submitted a report of the Nominating Committee which selected a regular ticket for offices for the ensuing year as follows: For President, Franklin Riffle; for Vice-President, H. D. Connick; for Secretary, Otto Von Geldern; for Treasurer, E. T. Schild; for Directors, Hermann Barth, Edward F. Haas, Hermann Kower, Carl Uhlig and C. B. Wing.

Mr. Marsden Manson presented the regular paper of the evening entitled, "The Struggle for Water in the Great Cities of the United States." Mr. Manson's paper presented an exhaustive study of the subject and will be discussed at the annual meeting of the Society, on Friday, January 18, 1907.

The annual meeting will be held on January 18th, on which occasion reports from the Secretary and Treasurer will be submitted and ballots for the new officers will be opened and counted.

The latter portion of the meeting was devoted to the discussion of San Francisco's water problem.

II. Annual Meeting, January 18, 1907

At this meeting the following officers were elected to serve one year: President, Franklin Riffle; Vice-President, H. D. Connick; Secretary, Otto von Geldern; Treasurer, E. T. Schild; Directors, Hermann Barth, Edward F. Haas, Hermann Kower, Carl Uhlig and Charles B. Wing.

The evening was devoted to an interesting discussion of Mr. Marsden Manson's paper entitled "The Struggle for Water in the Great Cities of the United States."

The next meeting of the Technical Society will be held either on the first or the eighth of February, at the Mechanics' Institute, San Francisco, when a second topical discussion on "Reinforced Concrete Construction" will be the business of the evening.

California Harbors—Further Action for Their Development

IN accordance with instructions given by the Counties Committee of the California Promotion Committee at its sixth semi-annual meeting, held in San Diego on December fifteenth last, at which fifty-one counties of the State were represented, the Committee on California Harbors met in Sacramento on January eleventh to discuss plans for the improvement of all the harbors of California with particular relation to the field on State legislature. The meeting was a most successful one, men of affairs from all parts of the State being in attendance, and the influence of the action taken will be wide spread.

The keynote of the meeting was embodied in a suggested concurrent resolution of the Senate and Assembly, which was unanimously adopted by the meeting. A copy of the resolution is appended below. It was also resolved to appoint a committee to wait upon Govenor Gillett and present to him the action taken at the meeting, and after the adjournment of the meeting the committee waited on the Governor and placed before him the resolution as adopted.

A committee was also appointed to see that the resolution was properly introduced in the legislature, and this was done on January 14, a concurrent resolution being introduced in both houses.

Harbor improvements are of vital importance to all parts of the State of California, and it is to be hoped that the resolutions of the Committee on California Harbors of the California Promotion Committee will meet with favor. We herewith print the Resolution in full.

Resolution adopted at meeting of the committee of California Harbors of the California Promotion Committee at Sacramento, January 11, 1907, and introduced in both houses of the Legislature, January 14, 1907.

"Resolution relative to the appointment of committees to investigate and report upon the conditions of the harbors of the State.

"WHEREAS, the constantly increasing commerce of the ports of California demands greater facilities for the handling of shipping, and

"WHEREAS, the interests of the entire State are affected by the harbors of the State, be it

"RESOLVED, that a special committee of ten members be appointed from the Legislature of the State of California, consisting of five members from the Senate to be appointed by the President thereof, and five members from the Assembly to be appointed by the Speaker thereof, to investigate the conditions of the harbors of the State and to make recommendations for legislation necessary to be enacted at this session to provide for present necessities and for future requirements.

"RESOLVED, that the Secretary of the Senate and the Clerk of the Assembly be directed to apportion such clerks and assistants to said joint committee from attaches of the respective houses as may be necessary for the prosecution of the work and the necessities of the joint committee.

"RESOLVED, that it is the sense of the Legislature that the report should be comprehensive in all respects and that it should include not only present conditions and facilities but also recommendations for improvements immediately necessary and also a plan for financing future permanent improvements.

"RESOLVED, that the said joint commtitee shall have the power to send for persons and papers and to issue subpoenas when necessary.

"RESOLVED, that the necessary traveling and incidental expenses of said joint committee shall be paid out of the contingent expenses of the respective houses."

Salvage in Class A Buildings When Gutted by Fire

A VERY interesting Bulletin, No. 124, has just been published by the National Fire Protection Association, 382 Ohio St., Chicago.

This bulletin gives the results of studies made by Mr. F. J. T. Stewart, associate member of the organization. Mr. Stewart is secretary of the National Fire Protection Association's Committee which compiled the report on the Baltimore conflagration, and as a member of the staff of the Continental Insurance Company, at New York, has arranged data on the percentage of cost for various items in buildings of Fire Proof construction.

Perhaps the most striking conclusion by Mr. Stewart is the statement that about 70% of a class A building is destroyed in a conflagration. We understand that similar studies now being made in San Francisco substantiate Mr. Stewart's arguments.

On January 15 and 16 the executive committee of the Board of Fire Underwriters of the Pacific Coast held their annual meeting at the Ferry building where many valuable papers were read, among them one by Mr. George M. Robertson. We are informed that Mr. Robertson in his paper gives parallel studies to those presented by Mr. Stewart. The published papers, which were read at this year's annual meeting of the Pacific Board of Underwriters, should be awaited with interest.

As Mr. Stewart's conclusions appear to us to be of especial import to our San Francisco readers at this time, we herewith present his table of figures and accompanying statement with due acknowledgment.

MR. STEWART'S STATEMENT

The accompanying table, showing how the cost of fire proof buildings is divided among the various items of construction, has been prepared from data furnished by architects and builders in the principal cities. As the analysis of the cost of construction was not uniform for all data received, some difficulty was experienced in an attempt to show a complete comparison. Thus, in some cases the cost of foundations has not been given and is probably included largely under mason work. The comparison is, however, practically complete in most cases as far as the five general subdivisions are concerned, and should be of value as indicating the amount of readily damageable material of a building in proportion to its total value.

Each column of figures in the table represents the data for an individual building, except the figures for New York, in the second, third and fifth columns, which show the average for a large number of buildings.

It has been demonstrated at Baltimore and San Francisco, that a first-class fireproof building exposed to a conflagration may be expected to sustain very heavy damage on all items of construction except foundations and steel frame. These two items represent approximately 20% of the value, and to this may be added about 10% for other salvage, chiefly mason work, thus giving 70% damage. This figure is close to the average loss, and less than the damage to a number of individual buildings in these conflagrations. Although it sounds paradoxical and discouraging to say that a first-class fireproof building may be damaged 70%, these buildings are to be encouraged, as they will retard and possibly limit a conflagration if properly grouped. This is especially true if all window openings are protected; side and rear window protection only is better than none. While an excellent fire department and numerous buildings equipped with automatic sprinklers are very essential nothing is, perhaps, a more effectual barrier to conflagrations than the grouping of fireproof buildings. They have demonstrated their ability to stop conflagrations when the fire can not pass at the side of them. The Horne Building in Pittsburg and the Granite Building in Rochester probably saved these cities from conflagrations similar to those at Chicago, Baltimore and San Francisco.

Only such fireproof buildings as those described in the standard specifications of the National Board of Fire Underwriters are contemplated in the above statement. So-called fireproof buildings, where the steel work is but partially protected or that are deficient in other essential features, will be likely to show a total loss as did the wholesale building of Silbey, Lindsay & Carr Co., in Rochester, during the conflagration of 1904. This building was of first-class fireproof construction, except that no protection was provided for the steel work. The additional cost of the protection would have been comparatively trifling.

TABLE SHOWING PROPORTION OF VALUE IN THE VARIOUS ITEMS OF CONSTRUCTION OF FIREPROOF BUILDINGS.

The figures opposite each item represent percentages of total cost of building.

LOCATION.	NEW YORK					CHICAGO					
Occupancy	Ofcs	Ofcs	Hot'l	Ofcs	W.H	Ofcs	Ofcs	Ofcs	Ofcs	Mer.	Mer.
Height in stories	10			12							
Cost cents per cubic ft.				40		30.3	34.8	32.3		14.00	23.6
FOUNDATIONS.						4.4	2.34	5.67		6.00	10.74
Excavations and Back Filling											
Shoring Banks, etc.											
Foundation Footing and Concrete											
Rubble Stone and Granite Pier Caps											
Sidewalks and Curbs											
STEEL FRAME	24.0	18.4	18.0	22.50	36.5	19.70	14.0	11.86	10.6	27.6	38.66
Material											
Erection											
Shop Drawings											
Painting											
Teaming											
MASON WORK	26.56	28.2	31.5	20.10	32.5	26.31	27.96	22.3	38.5	27.6	44.79
Brick—Common											
Brick—Face or Pressed											
Brick—Enameled											
Brick—Cleaning and Pointing											
Terra Cotta	1.41			2.06		10.04		5.76			
Stone	2.75			1.50			10.30				
Marble				10.90							
Wall Lining or Furring											
Floor Arches, Roof, etc.						11.06	9.16	5.8			22.08
Cinder Concrete Filling over Arches.						1.06	1.30	1.24			.47
Partitions											
Partitions—Cleaning and Wrecking											
Safety Deposit Vaults											
Miscl. Scaffolding and Wrecking											
Miscl. Masonry	22.40					4.15	7.2	9.5			22.24
EQUIPMENT	20.65	21.2	12.0	20.89	11.25	24.98	24.54	25.04	23.2	20.8	2.26
Elevator Plant	5.70			5.28		4.65	7.81	5.48			
Plumbing	3.49			4.16		6.58	5.90	6.19			.85
Heating System	5.85			5.85			.67	5.88			
Boiler Plant							1.92	1.86			.85
Lighting System, Wiring and Fixtures	5.37			5.6		3.82	2.76				
Dynamos, Switchboards, etc.								3.34			
Fixtures							.32	1.24			
Mail Chute	0.24					.24	.06	.13			
Filter Plant											
Refrigerating Plant											
Safes											
Vault and Safe Doors						.24	.29	.32			.56
Flash Signals and Indicators											
Furniture											
Ventilating						9.43	4.80	.60			
TRIM AND FINISH.	28.40	29.8	33.25	22.26	14.75	24.44	29.45	35.28	27.7	18.3	4.39
Carpentry—Rough						8.90	6.53	8.54			2.86
Carpentry—Finish	20.50			10.			1.88				
Hardware—Rough						.96	1.70	1.5			
Hardware—Finish											
Marble	2.83			9.85		5.02	4.17	7.61			.25
Mosaic	1.25										
Glass						1.02	1.32	3.37			
Slate											
Plastering	3.82			2.41		2.97	3.68	2.35			
Fresco						1.84	3.49	1.78			
Paint and Varnish						1.64	2.05	1.39			.30
Office Partitions (wood and glass)											
Ornamental Iron						1.09	2.58	6.19			
Skylights and Sheet Metal						1.00	2.05	2.55			.98
Office Grill											
GENERAL EXPENSES		4.4	7.70		5.00						

LOCATION.	BOSTON																
Occupancy	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Ofcs	Mer.	Mer.	Mer.	Mer.	Mer	Sch.	Stg.	Mer.	Mer.
Height in stories	9	11	10	10	11	11	11	11	7	4	6	3	5	3	7	5	6
Cost cents per cubic ft.	34.		35.	49.	67.	41.8	45.	34.			21.5	22.	29.	29.6			
FOUNDATIONS.	10.6	5.13	4.32	7.5	4.42	5.8			14.2	14.		11.5	13.5	7.4		24.5	
Excavations and Back Filling																	
Shoring Banks, etc.																	
Foundation Footing and Concrete																	
Rubble Stone and Granite Pier Caps																	
Sidewalks and Curbs																	
STEEL FRAME	14.4	27.4	22.5	15.	25.7	21.8	21.4	24.6	8.25	2.2	22.1	19.32	14.65	21.2	21.8	14.3	7.95
Material																	
Erection																	
Shop Drawings																	
Painting																	
Teaming																	
MASON WORK	40.4	15.0	32.23	41.6	33.60	27.57	34.85	26.	33.65	31.45	22.88	30.6	17.42	30.46	68.62	26.	38.6
Brick—Common	27.5	8.3	18.9	15.	19.7	17.75	17.7	16.9	29.2	30.	16.8	24.	9.75	20.1	64.	23.8	
Brick—Face or Pressed																	
Brick—Enameled																	
Brick—Cleaning and Pointing																	
Terra Cotta																	
Stone	7.1	1.75	8.17	16.6	5.25	3.55	10.4	4.54	4.45	1.45	3.3	6.6	6.95	6.6	4.62	2.14	
Marble	5.8	4.95	5.16	10.	8.65	6.27	6.75	4.56			2.78		.72	3.76		.08	
Wall Lining or Furring																	
Floor Arches, Roof, etc.																	
Cinder Concrete Filling over Arches.																	
Partitions																	
Partitions—Cleaning and Wrecking																	
Safety Deposit Vaults																	
Miscl. Scaffolding and Wrecking																	
Miscl. Masonry																	
EQUIPMENT	9.48	30.3	18.05	15.99	24.45	20.12	23.02	23.53	8.98	23.9	18.71	12.3	11.84	16.25	2.46	10.65	19.09
Elevator Plant	3.29	12.4	3.5	4.66	3.92	5.56	4.3	7.1	7.65	1.54		5.07	2.44	2.39		4.25	6.45
Plumbing	1.74	4.75	3.4	4.	4.8	5.45	2.92	5.47	.51	1.85	3.68	2.9	5.07	4.32		1.25	1.5
Heating System	2.71	8.4	4.83	5.33	2.88	2.48	8.46	7.8		9.85	5.5	3.34	2.34	7.64	1.54	1.36	2.52
Boiler Plant			1.4		5.68	2.37				10.4	7.15					1.07	
Lighting System, Wiring and Fixtures	1.74	4.75	3.79	2.00	1.62	4.26	7.34	3.16		.26	2.38	1.03	1.99	1.9	.92	1.5	7.2
Dynamos, Switchboards, etc.																	
Fixtures			1.13		5.55				.825							1.22	1.42
Mail Chute																	
Filter Plant																	
Refrigerating Plant																	
Safes																	
Vault and Safe Doors																	
Flash Signals and Indicators																	
Furniture																	
Ventilating																	
TRIM AND FINISH.	24.87	22.11	22.4	19.34	12.18	24.73	21.30	28.1	34.18	28.27	38.32	28.38	41.35	24.82	7.24	24.70	34.57
Carpentry—Rough																	
Carpentry—Finish	16.6	14.3	13.4	13.2	6.54	15.75	13.6	18.9	31.2	24.4	16.8	17.5	29.	15.1		19.9	30.8
Hardware—Rough																	
Hardware—Finish																	
Marble																	
Mosaic																	
Glass																	
Slate																	
Plastering	3.32	5.5	3.3	2.66	1.93	5.2	2.74	3.55	.572		12.6	2.0	6.7	4.3	1.54	.51	
Fresco																	
Paint and Varnish	2.9	1.65	4.	2.33	2.63	2.6	3.74	2.68	1.14	1.58	5.4	3.26	3.6	2.8	.30	2.86	2.34
Office Partitions (wood and glass)																	
Ornamental Iron																	
Skylights and Sheet Metal	2.05	.66	1.7	1.15	1.06	1.18	1.22	.986	1.27	2.29	1.52	3.6	2.05	2.72	5.4	1.43	1.43
Office Grill																	
GENERAL EXPENSES																	

LOCATION.	BALTIMORE								St LOUIS	
Occupancy	Ofc.	Ofc.	Ofc.	Ofc.	Ofc.	Ofc.	Ofc.	Ofc.	Ofc	Ofc.
Height in stories	11	12	7	14	9	8	10	7	12	12
Cost cents per cubic ft.										
FOUNDATIONS.	5.62	4.37	7.25					4.3	5.	7.
Excavations and Back Filling	1.22	1.25	2.07					4.3		
Shoring Banks, etc.		1.06								
Foundation Footing and Concrete	4.4	.74	5.							
Rubble Stone and Granite Pier Caps		.65								
Sidewalks and Curbs		.13	.18							
STEEL FRAME	13.8	14.54	19.52	9.9	9.	10.1	11.4	17.5	15.0	15.0
Material		11.79								
Erection		1.7								
Shop Drawings		.49								
Painting		.24								
Teaming		.32								
MASON WORK	23.76	28.5	34.15	30.15	31.31	37.2	29.9	35.5	20.0	25.0
Brick—Common	10.1	6.59	11.21				8.98	13.5		
Brick—Face or Pressed		1.82	1.26							
Brick—Enameled		.86								
Brick—Cleaning and Pointing		.21								
Terra Cotta	4.34	7.61	5.07	3.9	2.5		3.5			
Stone	2.55	4.22	9.34	4.05	7.4	27.3	8.7	9.4		
Marble					.83					
Wall Lining or Furring	.77				1.8					
Floor Arches, Roof, etc.	3.87	4.73	3.68	6.8	6.6	3.4	3.6	6.6		
Cinder Concrete Filling over Arches.	.57	.58	.49					6.		
Partitions	1.56	1.88	3.1							
Partitions—Cleaning and Wrecking										
Safety Deposit Vaults				7.6			5.16			
Miscl. Scaffolding and Wrecking					.48					
Miscl. Masonry				7.8	11.70	6.5				
EQUIPMENT	19.18	24.54	15.96	18.69	26.71	15.23	21.23	12.89	17.0	18.0
Elevator Plant	7.6	7.68	8.15	5.	5.68	3.8	7.7	4.02		
Plumbing	3.26	3.55	2.61	4.13	3.5	3.7	3.4	3.36		
Heating System	4.6	7.06	3.54	4.2	3.6	2.2	6.7	4.15		
Boiler Plant					1.9					
Lighting System, Wiring and Fixtures	2.54	5.05	1.38	2.03	1.9	1.6	3.	1.16		
Dynamos, Switchboards, etc.					2.18					
Fixtures	.7	.49		1.05	.65					
Mail Chute	.48	.27	.28	.18		.33	.43			
Filter Plant		.44								
Refrigerating Plant				.93						
Safes					1.9					
Vault and Safe Doors				.63		3.6				
Flash Signals and Indicators				.54						
Furniture					5.4					
Ventilating										
TRIM AND FINISH.	36.08	27.58	22.83	36.20	28.78	38.63	26.89	25.04	33.0	27.0
Carpentry—Rough	2.03	2.59	.22	10.9	9.8	5.4	7.9	1.6		
Carpentry—Finish	5.27	4.31	.51					5.7		
Hardware—Rough	.32	.29	1.7		.64	.9	1.16	1.95		
Hardware—Finish	.74	.88	5.1	.83						
Marble	6.74	7.11	1.42	10.5	5.4	10.1	6.2	2.8		
Mosaic	.14		1.99			1.2				
Glass	1.44	9.	1.13	1.4	7.5	1.4		.89		
Slate										
Plastering	2.94	3.05	2.96	2.9	4.21	3.7	2.7	1.4		
Fresco	.88									
Paint and Varnish	2.00	1.34	1.45	2.9	1.6	1.4	2.9	1.5		
Office Partitions (wood and glass)	2.65									
Ornamental Iron	9.70	5.87	4.62	6.4	5.	10.5	5.5	6.8		
Skylights and Sheet Metal	.71	1.22	1.54	.37	.63	.83	.53	2.4		
Office Grill	.5		.19			1.2				
GENERAL EXPENSES	1.83	1.92		5.17	4.31	.36	11.2	6.94	10.0	10.0

Abbreviations: Ofcs.—Offices; Mer.—Mercantile; W. H.—Warehouse; Sch.—School; Stg.—Storage.

Building News and Trade Notes

WE shall welcome any news notes that our readers may send to us. Notes of contracts awarded, bidding prices, etc., will be especially acceptable. While this journal devotes itself most decidedly to architectural construction, it wishes to reflect also the activities of engineering in its various phases, especially civil engineering in its relation to the design and construction of large and important works. We shall be pleased also from time to time to make statements bringing to the attention of our readers the equipment or development of manufacturing plants of interest and worthy of notice. We shall likewise be glad to refer to manufacturing enterprises and manufactured articles, especially relating to structural products for building work.

Oil, Oil Burners, and Oil Burning*

By Staples and Pfeiffer

WITH themarvelousdevelopment of the California and Texas oil fields and the ease and modern facilities of transporting oil from the wells to the furnace, together with the saving in cost of fuel, space and weight are the principal reasons for the recent changes in our steam plants, vessels, railroads, breweries, bakeries, hotels, laudries, and, in fact, all description of steam and electric plants from burning coal to that of fuel oils.

This change has been made, too, at a saving of from 25 to 75 per cent, depending on the distance from the source of supply, the apparatus used and the sensible and economical hanlding of same. This transition is certainly an agreeable one, from dust, dirt, ashes, soot and smoke to cleanliness, labor-saving and a more general air of prosperity around the plant.

Practically all the California and Texas fuel oils contain more or less water, sand, asbestos, fibre, marsh gas, etc.; some grades flowing free, while others are more tenacious, though of lighter gravity, still others are a mixture of light and oils, forming and carrying an oil gas in solution, causing trouble at the pumps.

In general:

The gravities range from 14 to 20 Beaume.

The heat units range from 16,000 to 22,000.

The per cent of sulphur from 1 to 3.

The per cent of water from 1 to 5.

The per cent of steam to atomize from 2 to 5 per pound of water evaporated.

The per cent of air to atomize from $\frac{1}{4}$ C. F. at 20 pounds pressure per pound of water evaporated.

The per cent of air to atomize from 4 C. F. at 1 pound pressure per pound of water evaporated.

One gallon of oil weighs 7.6 pounds; 1 barrel of 42 gallons equals 320 pounds, $3\frac{1}{2}$ to 4 barrels or 1,200 pounds, or 24 C. F. equal 1 ton of coal. A saving of 50 per cent in weight, 75 per cent in space, and possibly 25 to 75 per cent in money over coal.

A neat, compact, self-contained oil-pumping system, installed by practical mechanics or firm of good reputation is a continued source of pride and a pleasure to handle, combined with the positive, instant and absolute control of the fire, from its start, to the highest efficiency of the boilers. The oil heater should be of cast iron with flanged head, body of heater tapering off, forming a firm base by which to fasten it down, the base at the same time acting as a settling chamber to blow off all dirt and water which will precipitate from the heating of the oil. The upper portion of heater carrying drip pan at least twice the capacity of oil end pumps, an important point in a rush overhauling job.

Two duplex pumps fastened in drip-pan, pumps fitted with split rings, metal valves for oil lift, lead or fibre gaskets, brass cross connections in one piece with valves and ground joint unions next to pumps so one pump can be overhauled while the other is in use; pumps to discharge to air chamber, and through heater to burners.

Self-cleaning strainer, oil meter; thermometer and by-pass on line, if desired; a special Automatic Regulating relief valve or pump governor, or both, are good investments towards a steady oil feed to the burners.

The air chamber must be absolutely tight, having no air cock, valve or plug in or near the top to let the air escape and chamber fill with oil. A small air relief pipe from bonnets of pumps is also appreciated if oil has gas in solution in starting up. Have an extra heavy copper coil fitted to and through flanged head of heater, thus having no joints to cause leaks inside of heater.

Exhaust of pumps or live steam should pass through copper coil in heater to heat oil in heater, exhaust of coil to feed water heater, condenser or atmosphere as desired.

All valves on suctions from tanks, discharge, bypass, relief, strainer, pump governor gauges and all details to be next to pumps, and not scattered here and there.

Steam to burners must be dry and of steady pressure; if Air is used, be sure to have compressor large enough to save running at high speed, having large receiver to insure a steady pressure.

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*Reprint from the Pacific Mining and Oil Reporter, Vol. VIII No. 5, page 28, January, 1907.

Before starting up be certain that all pipes are thoroughly blown out, although the scale and dirt in a new system may cause trouble for a week or more. As to furnace arrangements and burners, that is a delicate question; all having different views and ideas.

A proper burner will superheat, pulverize and vaporize the steam and oil through a series of mixing chambers into an Oil Gas before it leaves the slotted tip, the oil gas lights and burns close to tip with a soft, clean, steady flame, just the same as a large gas jet and is as easily controlled by one valve, and is self-cleaning in its action.

As such, a burner has no ejector, injector, siphon, steam-jet or blow-pipe action, the combustion is in the furnace, not in the stack. Being a pure oil gas fire it causes no leaky tubes or seams, and gives the highest efficiency without the need of superheating or extension fronts, brick or checkerwork, etc., as the burner is complete in itself, and the furnace arrangement is simply to control the free air admitted for combustion.

In starting up, open dampers, which should be fitted to fall open, not closed, as a safeguard in case of accident with it, often bad results. Start pump to circulating oil; blow out the burner and leave steam on lightly to warm it up; place lighter in furnace near burner tip, open oil valve slightly, and as the oil gas ignites regulate valves to suit fire wanted, withdrawing the lighter and regulating ash pan damper for proper amount of air for combustion.

If too much steam or air is used with which to atomize it will show a short, stringy, roaring fire. If too little steam or air is used, it shows in a dull, dead, smoky fire. Too much air at ash pan damper shows a sparkling light to bluish smoky flame, with a tendency to blow up and away from air inlet.

Too little air at ash pan damper causes the dull, dead fire, again with the addition of shaking and drumming of doors, breeching and stack, and a possible explosion of unconsumed gases with its natural results.

As to the effect of oil on boilers, that depends on details. If your tube ends are burnt off, or they and the seams leak, it is the fault of the burner that plasters your furnace and combustion chamber full of unconsumed oil, dripping in the ash pit, glazing and melting down the brick side walls or rings, choking and gumming heads, tubes and stacks with an oily soot that blisters, pits and corrodes through the unconsumed sulphur which forms sulphuric acid with the water in the oil, and causes what is called "hot short," that possibly being the cause of your trouble, especially if the air for combustion is not under control, but simply passes into and through the furnace, combustion chamber, tubes, and up the stack, without once passing through the fire, but forcibly cooling what you are really trying to heat and straining the boiler by the uneven temperatures at the different points.

Swing joints fitted to burners set in fire doors, as in the fast Vallejo steamers "Arrow," "General Frisbie," steamers "Monticello," "Iaqua," "San Gabriel," "Brunswick" and "James H. Higgins," are highly appreciated by marine men at all times.

The remark is often made: "There is no difference in burners," "the heat units are not in the oil," "we have perfect combustion, as our stack shows no smoke," etc., but let us reason and see what Admiral Melville of the U. S. Navy says in his report: "Out of some 2,000 burners submitted, over 90 per cent were worthless," quite a difference, and again in a series of fourteen tests under a Hohenstien water tube boiler, the reports run from 10.77 to 14.43 F. & A. 212 degrees.

In the U. S. Government test in the new San Francisco Postoffice under a Heine Boiler by the Government experts an evap. or 15.8 F. & A. 212 degrees.

From a number of tests at the Union Mills with different burners the reports range from 12.44 to 16.50 F. & A. 212 degrees under a Heine water tube boiler.

At the American Steel and Wire Works in a seven days' test the burners ranged from 14.50 to 17.35 F. & A. 212 degrees under a tubular boiler.

The American Beet Sugar Company report tests under a water tube boiler of from 14 to 17 F. & A. 212 degrees. At the Western Sugar Refining Company in over 25 burners tested the reports run from 6½ to 14.20 F. & A. 212 degrees. Under an English gunboat type of Scotch marine boiler..

All burners tested under the same conditions were burners of the injector, ejector, siphon, blow-pipe, steam jet and Automatic Oil Gas types, flat flame, round flame, inside and outside mixers and oil gas mixers, showing conclusively that there is a difference in burners, just as there is in men, in sunshine and rain, in day and night, and in diamonds and paste.

WALL COVERINGS

With the reorganization of San Francisco, architects and owners will be interested to know that there is on the market a new indestructible, washable wall covering called Sanitas.

These goods are put up in rolls 45 inches wide and 12 yards long, and are excellent decoration for bathrooms, lavatories, kitchens, pantries, etc. These goods are made in various designs, in dull and glazed finish, are easily hung, and can be washed with hot or cold water. They are also recommended for store and office purposes; special designs and colors are made up for this use. Uhl Brothers, 721 Market St., San Francisco, are the Pacific Coast agents, and they will gladly furnish samples and further information upon request.

AMERICAN BVILDERS REVIEW

SAN FRANCISCO, MARCH, 1907

A VIEW AT STANFORD UNIVERSITY

WATERHOUSE & PRICE COMPANY

BUILDING MATERIALS

HANDLE

FOR GENERAL USE

Richardson's Fireproof Doors and Finish (used throughout the Kohl Building).

"Bar Lock" Sidewalk Light Construction.

"Paschall" Sidewalk Light Construction.

Metile (Metal Tile for Walls and Ceilings).

Merritt's Expanded Metal Lockers (for Schools, Factories, etc.)

Cabot's Sheathing and Deafening "Quilt."

Building Papers, Asbestos and Felts.

Hill's Clothes Dryers (for Yards and Balconies).

Ornamental Decoration; Composition and Plastic; Interior and Exterior; Caps, Brackets, etc.

Structural Steel.

Tyler's Ornamental Iron.

Van Dorn Steel Joist Hangers.

Van Dorn Steel Post Caps and Bases.

Bruening Cork Insulating and Cork Boards for Cold Storage and Brewery Buildings.

American Three-Way Prismatic Glass.

Grant Overhead Pulleys.

'Peerless" Sheet Rubber Tiling.

"Columbus" Steel Rolling Shutters.

"Flexifold" Wood Rolling Doors and Shutters.

Revolving Doors.

Sanitary Clothes Dryers (for Residences and Apartment Houses).

Church Pews and Opera Chairs.

Pneumatic Tubes.

Twisted Steel Bars.

Lock-Woven Fabric.

Interlocking Rubber Tiling.

Mead Gas Logs.

Mead Steam-Gas Radiators.

Asbestos Theatre Curtains.

FOR MASONS' USE

"Acme" Lime.

"Indian" Cement.

"Lafarge" Non-staining Cement.

Victoria Brand Keene's Cement.

Welsh Quarries (Tile).

Cabot's Mortar Color Pulps.

Cincinnati Metal Wall Ties.

Whalebone Metal Wall Ties

Rutty Metal Wall Plugs.

Tiffany Enameled Brick.

"Never Fade" Dry Mortar Colors.

FOR ROOFERS' USE

Taylor's Roofing Tin, Redipped.

Taylor's "Target & Arrow" Old Style Roofing Tin.

Taylor's "Columbia" Extra Coated Tin.

Common Terne Roofing Tin.

Marble Faced Ready Roofing.

Induroid Ready Roofing.

Metal Spanish Tiles.

Rubber Sanded Ready Roofing.

FOR PLASTERERS' USE

Ivory Cement Plaster.

Ivory Wood Fibre Plaster.

Gold Medal Finish.

Universal Finish.

Superfine Casting Plaster.

"Lion" Plaster Fibre.

Alpine Plaster.

Acme Lime.

Herringbone Expanded Steel Lath.

Herringbone Truss Lath.

Sackett's Plaster Board.

Union Metal Corner Bead.

Hannon Metal Corner Bead.

FOR PAINTERS' USE

Cabot's Creosote Shingle Stain.

Cabot's Brick Preservative.

Rinald's Porcelain Enamel and Porcelain Paint.

Bessemer Paint for all Metals.

Superior Graphite Paint.

Rinald's Galvanic Primer

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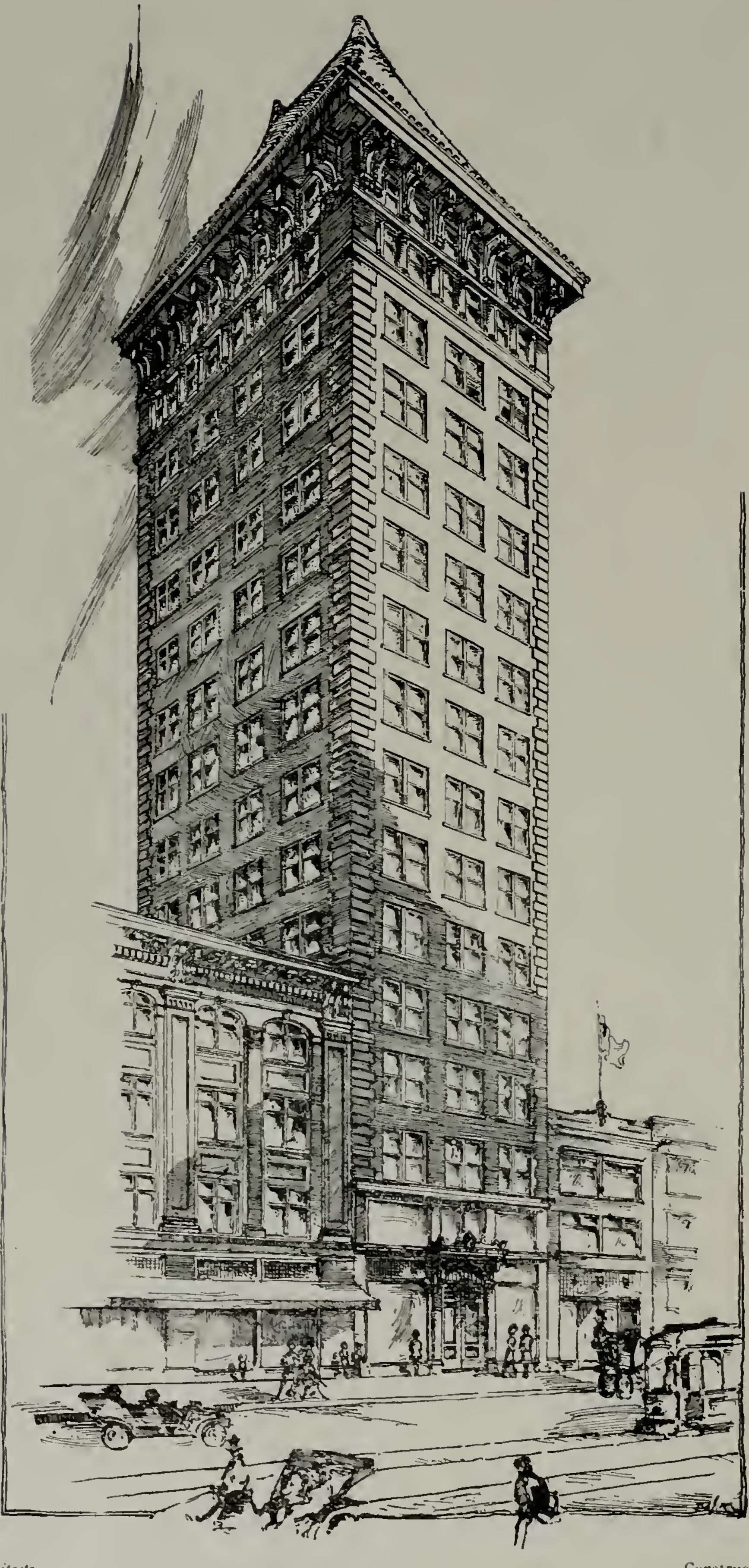


Shea and Shea, Architects.

WHITTELL BUILDING
San Francisco

Constructed by J. R. Bowles

AMERICAN BUILDERS REVIEW

VOLUME VI MARCH, 1907 NUMBER 3

The Whittell Building

SAN FRANCISCO

IN the history of the rehabilitation of the city of San Francisco, the Whittell building will occupy a very conspicuous place. The foundations of the structure were completed several months before the catastrophe of April 18, 1906. The erection of the steel frame was begun about the first of February of that year, and, as the accompanying erection views show, the assemblage of the complete steel frame was substantially complete on the 9th of April. The structural frame was, naturally, severely tested by the earthquake vibration; and, at least in its lower stories, greatly heated by the fire in adjoining buildings; yet the steel frame was little harmed, and the view (Figure 6), taken on May 15, 1906, shows the structure as it appeared within one month of the fire. Soon afterward reconstruction was begun where necessary, and the building is now in process of rapid completion.

The Whittell Building is situated on the north line of Geary Street, about 80 feet east of Stockton Street. Considering the height of the building, its plan is quite small, being 45 feet front by 60 feet in depth. The structure rises to a height of 17 stories. The main cornice is a few inches more than 200 feet vertically above the sidewalk.

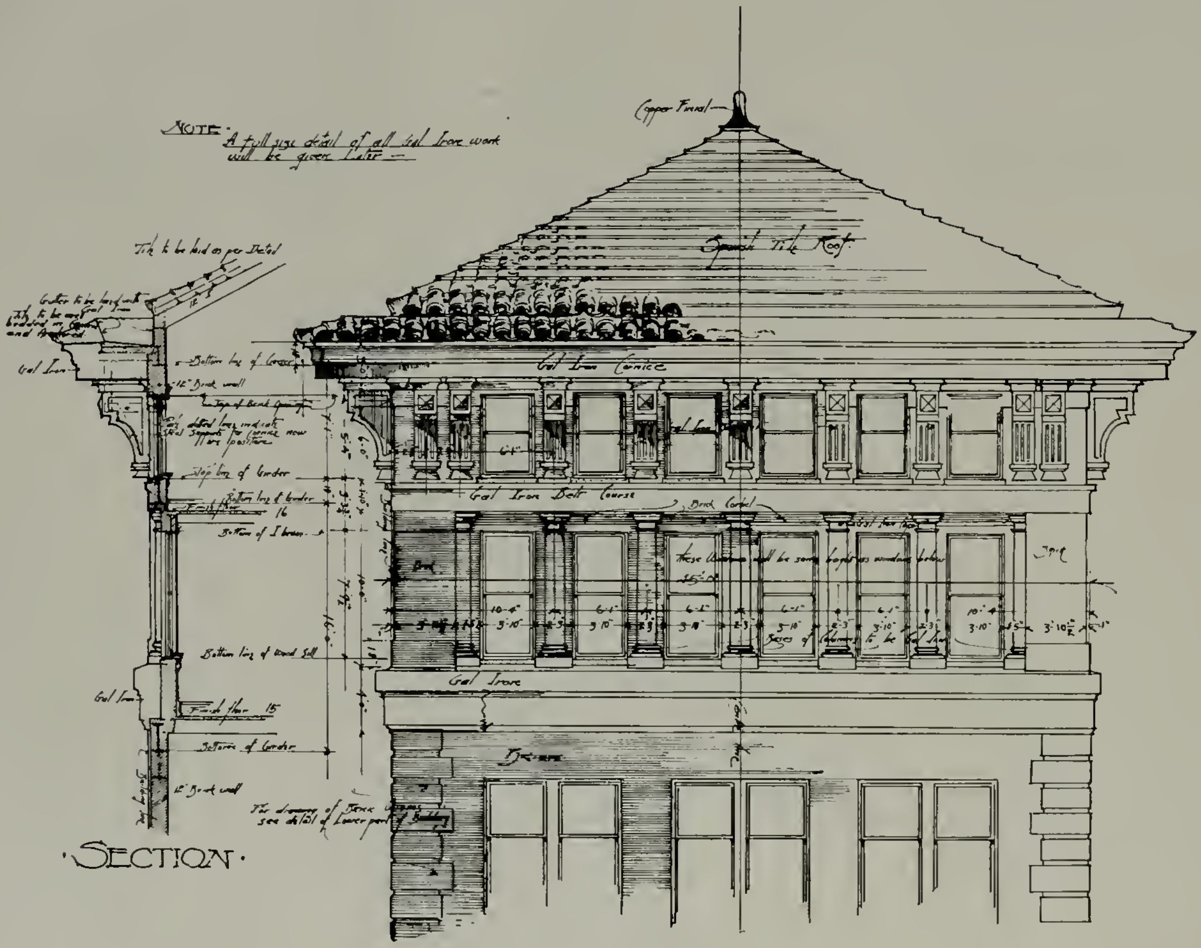
This substantial building, which forms one of the most striking structures in the nucleus of sky-scrapers in the business district, is being built for Mr. George Whittell, one of San Francisco's capitalists. The estimate for the total expenditure is $450,000,00. The architectural features were designed by Shea & Shea, architects. The J. R. Bowles Company, of Portland, Oregon, have designed the structural frame and foundation, and the same company is erecting the building. The construction is being superintended in all its parts by Mr. George A. Burch, the local representative for J. R. Bowles.

The Whittell Building has become known as the "Bird Cage" structure, and immediately after the fire occupied a most conspicuous place among the devastated ruins of the city, as it could be seen on every hand, rising, like the Call Building, to the sky. In fact, it still occupies, in that regard, a unique position.

At the time of the earthquake and fire, the steel frame was about completed. The damage from the earthquake was very slight. The designers attribute this small damage to the character of the foundation. As described later, it will be seen that the foundation is a substantial unit and at considerable depth below the street level. There is, further, great mass to the foundation, and the designers argue that these features of the foundation, its integrity, its depth, and its mass, develop for the small area and great height of the building a buoyant principle and bond between foundation and superstructure, such as is used on a small scale in ingenious toys, which appear top-heavy but which always return to an upright position when tilted, because of the mass at the base.

These structural features of foundation and the general scheme for stability are due to Mr. J. B. C. Lockwood, C. E., Mem. Am. Soc. C. E., who has designed many prominent structures.

The excavations for the building extend down to "hard pan," about 24 feet below the curb line, as shown on the accompanying structural drawings. A solid bed of concrete about 5 feet 6 inches thick was laid over the entire area. More than 600 barrels of cement



WHITTELL BUILDING
Top Details

were used for this purpose. The foundation grillage, as shown by the grillage plan, contains about 150 tons of steel, mainly in the form of 24-inch, 18-inch and 8-inch I beams, thoroughly imbedded in solid concrete.

Upon this foundation the superstructure was reared in the extremely short time of eleven weeks, making what is believed to be one of the speediest erection records to date. The erection of the steel work was performed by the firm of Long & Hoyt, of San Francisco. The despatch with which the superstructure was erected was partly due to the equipment used. Pneumatic tools were installed and employed on the entire erection of the superstructure. Not a single accident occurred during this erection. The superstructure contains about 883 tons of steel. Of this steel work, the most admirable feature is a system of wind bracing. This bracing is mainly developed by the use of spandrel plate girders at the floor levels, which girders run around the rectangular frame, as is clearly shown by the accompanying illustrations. A similar form of bracing may be noted in the Humboldt Bank Building, on Market Street near Fourth. Lattice girders, however, are mainly used in this case, as against plate girders.

That spandrel steel girders form a well-founded and substantial system of lateral bracing was certainly demonstrated on the Whittell Building by the recent earthquake vibration, which , as already mentioned, left the structure in a perfectly safe condition. Of course it should be noted that the frame was not encumbered with a heavy dead load of walls. Yet the experience of the Call Building must substantiate the claim for the advisability of using spandrel girders. In the Whittell Building after the earthquake, no fractures in the massive concrete foundation could be detected.

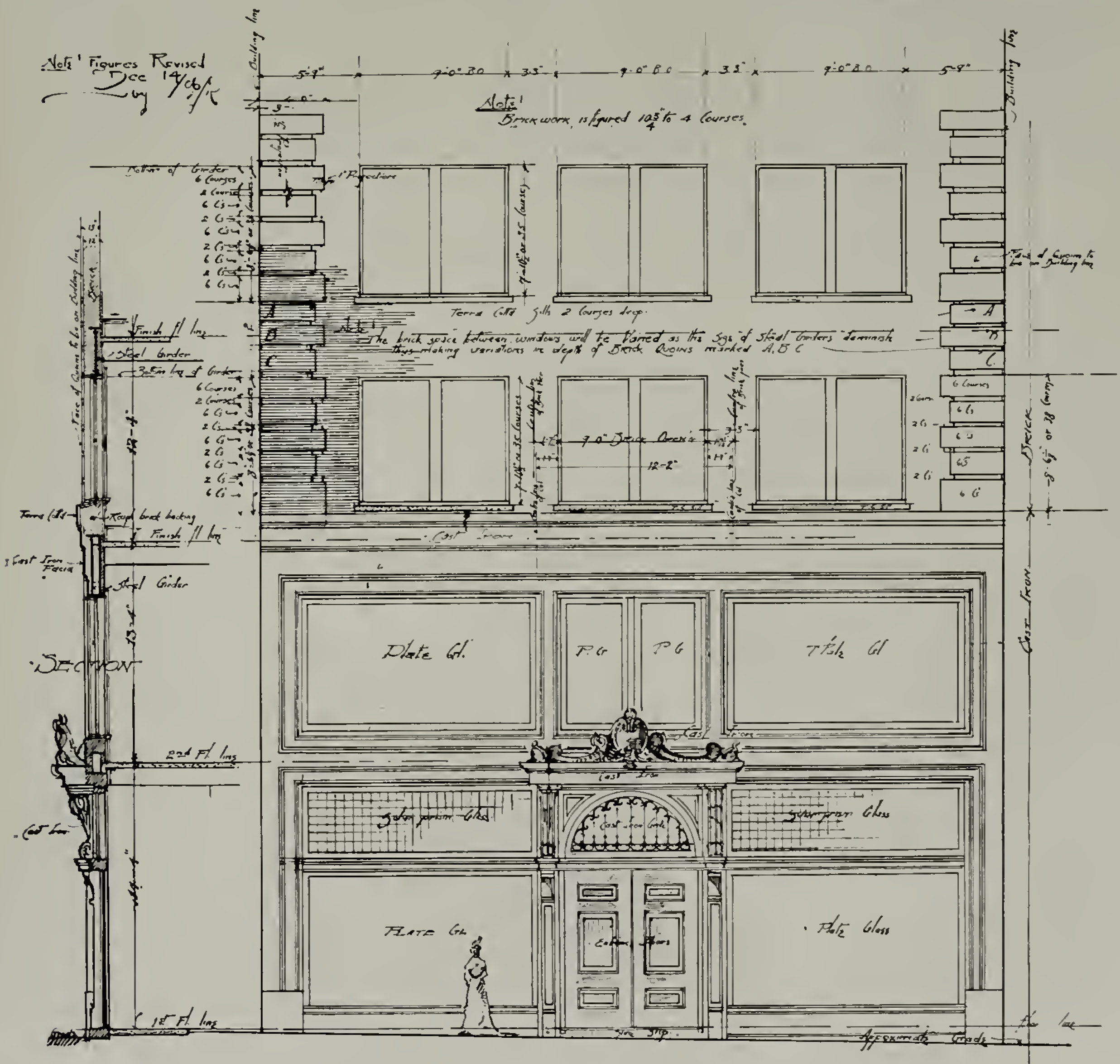
At the time of the disaster a great deal of false work had been installed in the structure from the ground to the fifth story, and, together with other light but inflammable material which the building contained, there was enough material in the lower portion of the building to create a fire of high temperature. Moreover, the surrounding buildings on three sides were of very inflammable nature, and all this helped to subject, at least, the lower stories of the Whittell frame, to a considerable heat. Notwithstanding, the damage to the steel frame was slight, and consisted of a small amount of torsion of column No. 14, on the first floor; some warping in girder No. 5, on the second floor; and similar warping in girder No. 66, on the third floor. Of course the painting of the steel frame also was damaged.

The architectural design is of English treatment in detail and will be carried out as shown by the illustrations. The roof will be covered with a red Spanish tile. The walls will be of yellow brick.

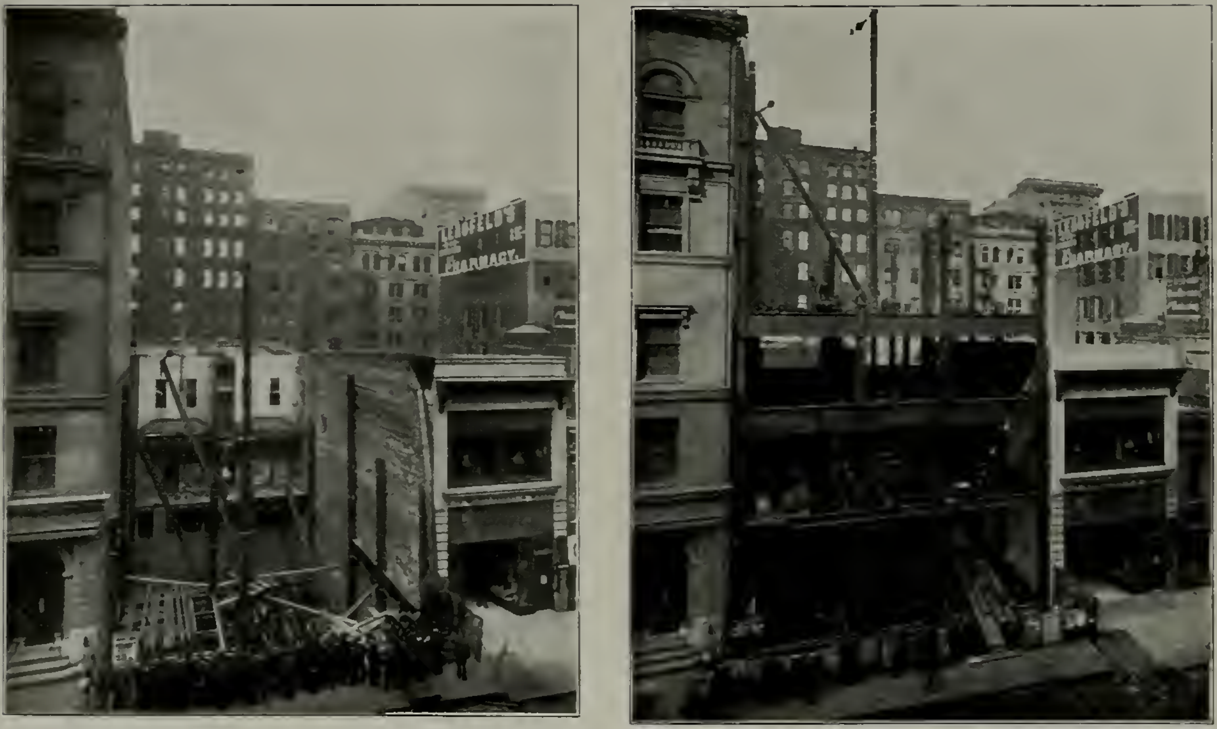
The building has been designed as a loft building, but in such a manner that it may easily be made into an office building. Two Otis elevators will run the full height of the structure, in addition to the stairway. The ground floor is to be used for stores. It is estimated that the building will be completed for occupancy about September 1, 1907.

The Clinton system of fire proofing has been installed, which, as is well known, consists mainly of a welded fabric made of No. 6 and No. 10 gauge wire, galvanized and formed into

Continued on Page 78



WHITTELL BUILDING
Front Elevation, Showing Entrance and Details of Brickwork



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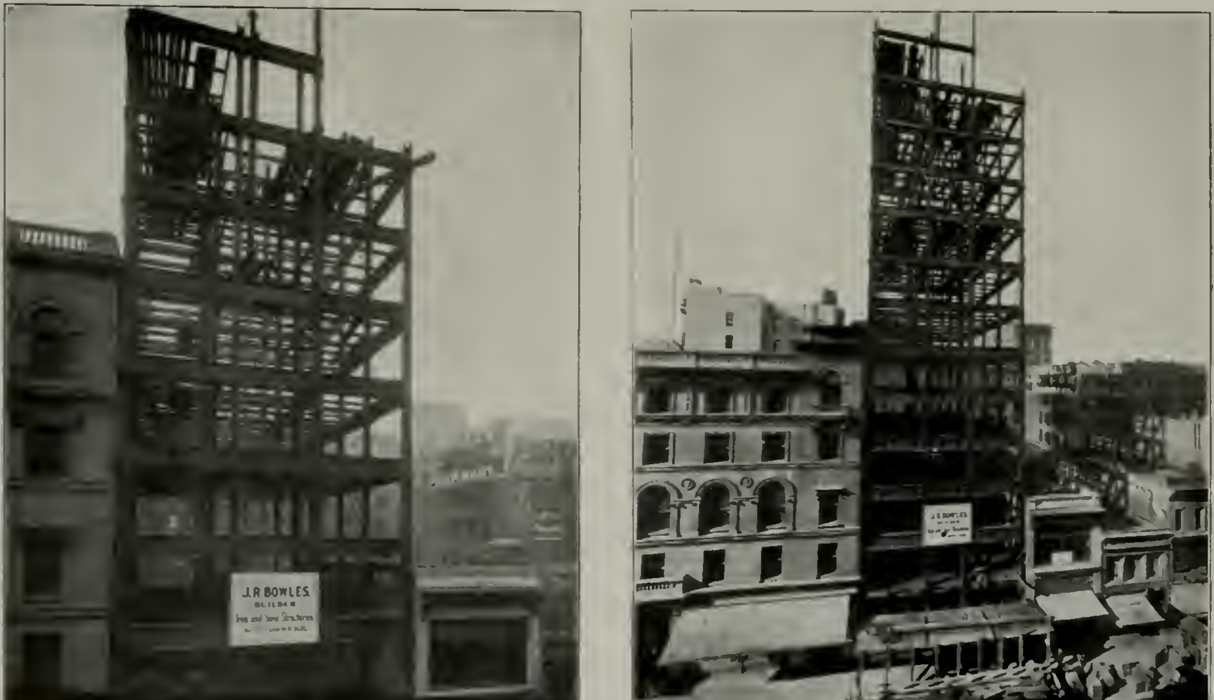
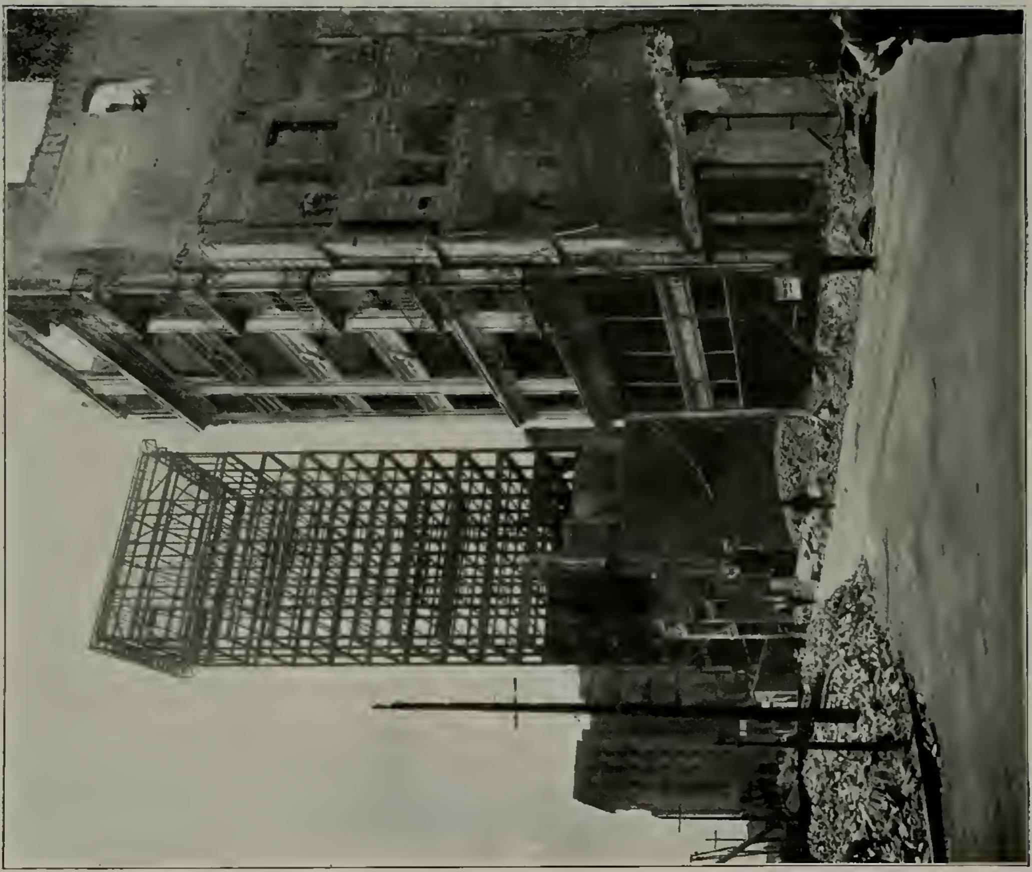


MARCH 12. 1906

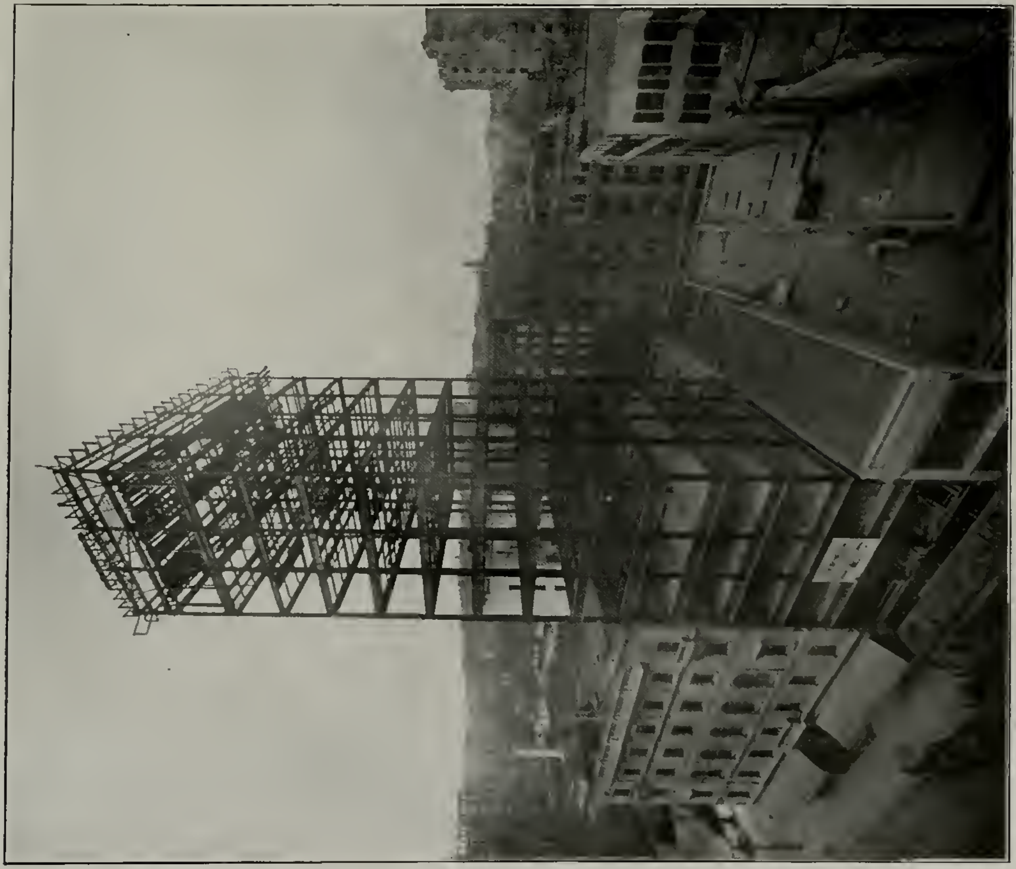
MARCH 26. 1906

WHITTELL BUILDING

Progress Views; Erection of Steel Frame



WHITTELL BUILDING
May 15, 1906



WHITTELL BUILDING
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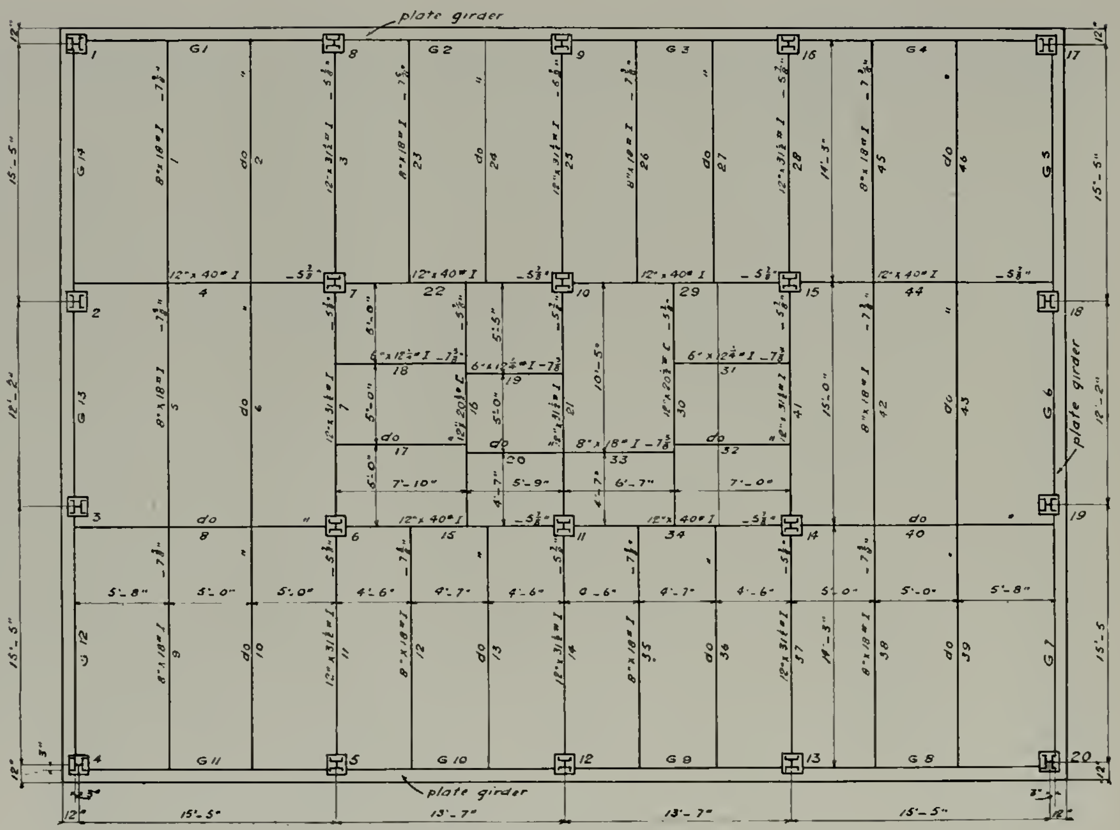
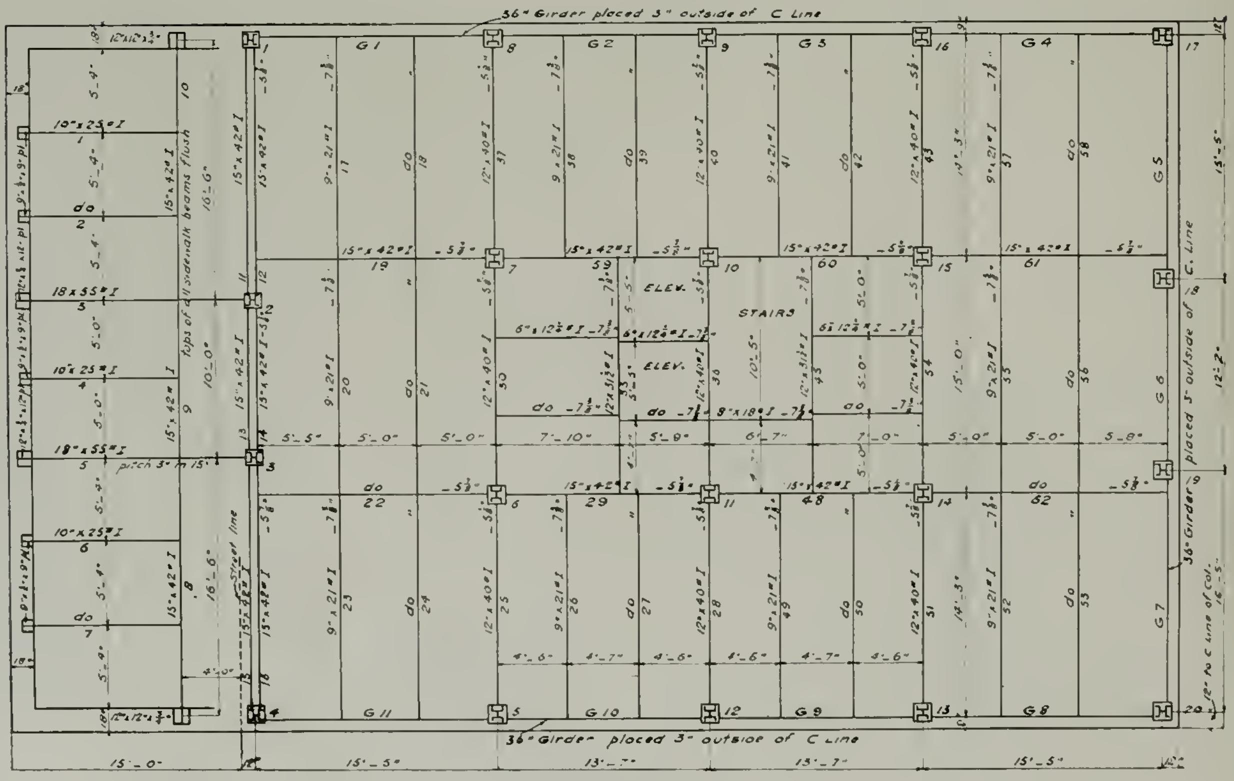
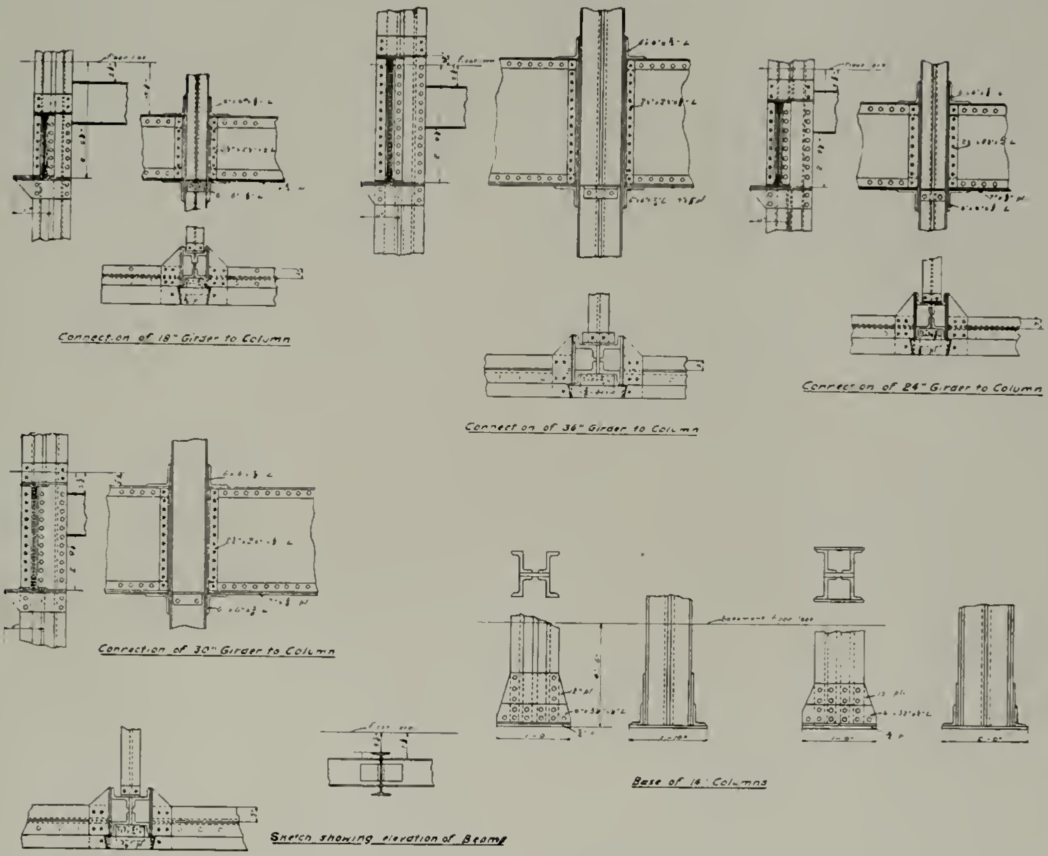


WHITTELL BUILDING
Typical Floor Framing Plan



WHITTELL BUILDING
First Floor Framing Plan


Connection of 18" Girder to Column

Connection of 36" Girder to Column

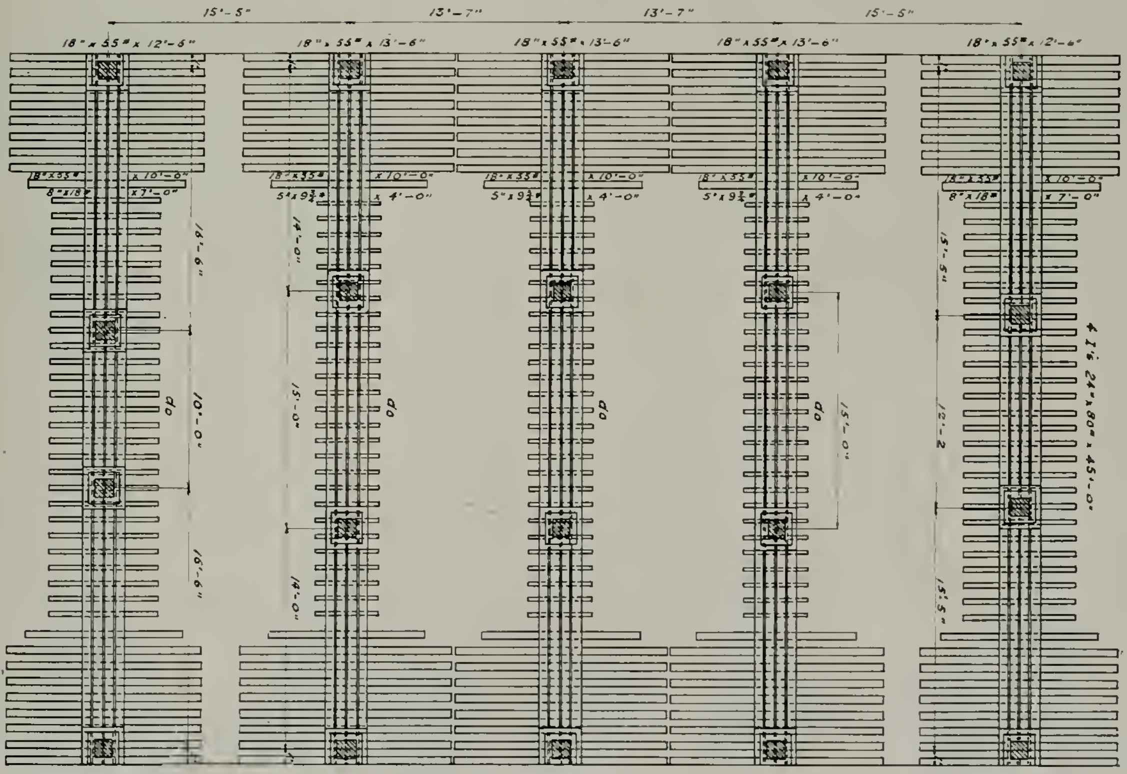
Connection of 24" Girder to Column

Connection of 30" Girder to Column

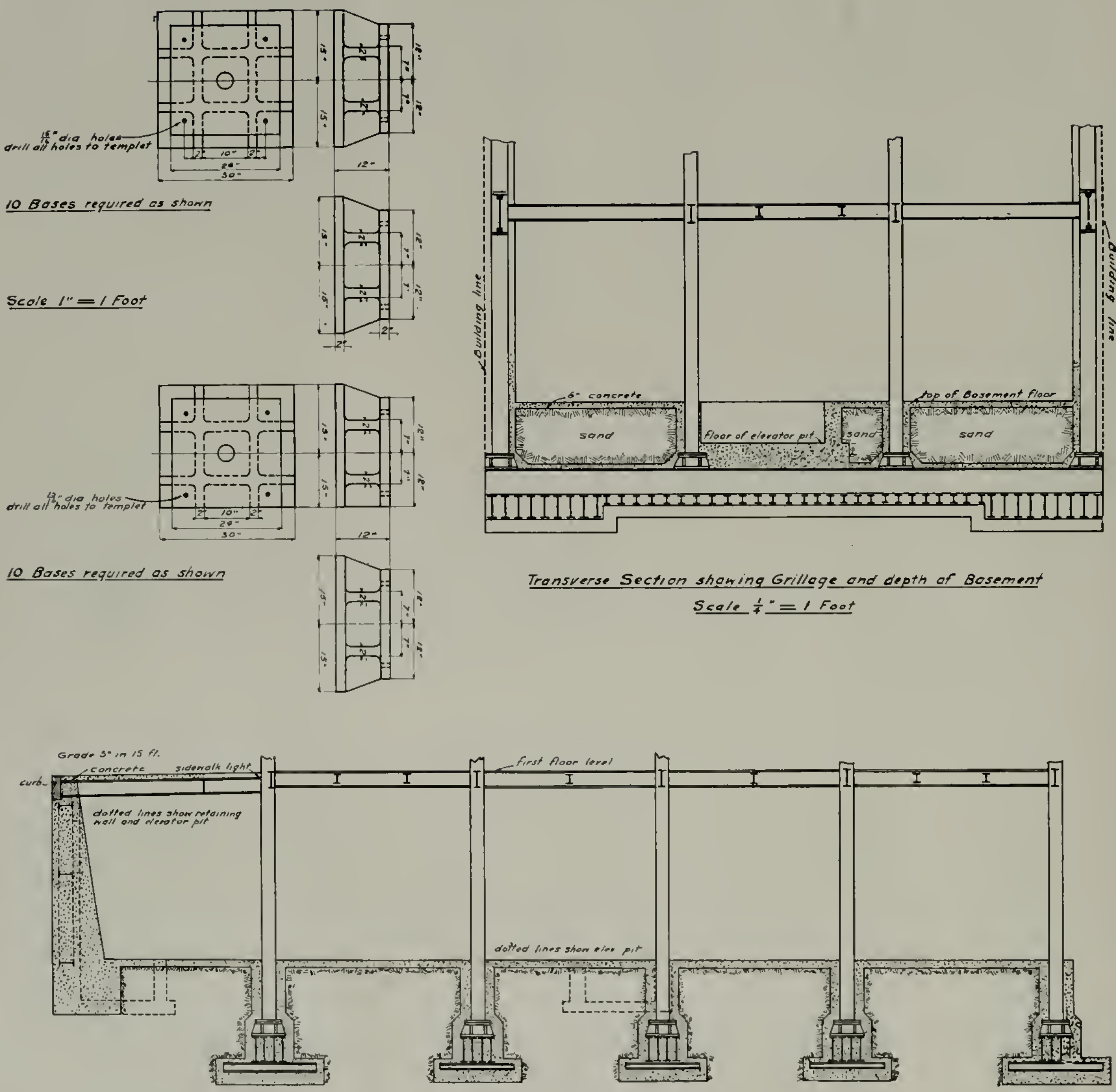
Base of 14" Columns

Sketch showing elevation of Beams

WHITTELL BUILDING
Structural Steel Details



WHITTELL BUILDING
Plan of Foundation Grillage



$\frac{15}{16}$" dia holes drill all holes to templet
2" 10" 2"
24"
30"
15"
15"
18"
18"
7"
7"
12"
2"
10 Bases required as shown
Scale 1" = 1 Foot
$\frac{13}{16}$" dia holes drill all holes to templet
10 Bases required as shown
Building line
Building line
6" concrete
top of Basement floor
sand
Floor of elevator pit
sand
sand
Transverse Section showing Grillage and depth of Basement
Scale $\frac{1}{4}$" = 1 Foot
Grade 5" in 15 ft.
concrete
sidewalk light
curb
first floor level
dotted lines show retaining wall and elevator pit
dotted lines show elev. pit

WHITTELL BUILDING
Foundation Plans

electrically welded rectangles of dimensions of 3 x 8 inches. The fabric has been laid through the entire length of the structure, so as to form a continuous bond for the concrete for the full dimensions of the building. The fire proofing has been carefully attended to, and every precaution has been taken to make it complete in every detail. The entire structural steel frame has been covered with concrete.

We are indebted to Mr. George A. Burch and his assistant, Mr. C. B. Rushmer, of the local staff of the firm of J. R. Bowles, for the material from which this article has been prepared. The accompanying drawings will give a substantial idea of the structural design and the architectural features of this excellent addition to the first class buildings of reconstructed San Francisco.

Building For the Western Meat Company

Sixth and Townsend Streets, San Francisco, Cal.

BEFORE the disaster of last April there was little reinforced concrete work for buildings prosecuted within the city limits of San Francisco; (1) because the type of construction was new and not well understood; (2) because the building ordinance of San Francisco in its prescriptions was not favorably disposed toward the use of concrete and steel for buildings.

Since the fire, however, the situation has greatly changed. The new building ordinance is decidedly more favorable in its requirements and recognizes reinforced concrete as a legitimate and proper class of construction. Moreover, there appears to be an ever-increasing demand on the part of owners for buildings of reinforced concrete, and consequently such structures are being erected in all parts of the burned district.

The combination of concrete and steel lends itself readily to variation of treatment, especially in regard to the form of the reinforcing metal, and therefore many different types or systems of reinforcement have been advanced at the present time in San Francisco. A large number of these different types or systems of design may at present be studied to advantage while buildings are under course of construction.

All of these different systems have more or less merit; naturally some are better than others. Reinforced concrete construction is still in its infancy, and whether or not a particular system shall live and continue to be used will depend greatly upon two conditions which must be satisfied: (1) relative cheapness in cost of construction; and (2) sound design, embodying the principles of mechanics to a rational extent in the calculation of stresses and the proportioning of parts.

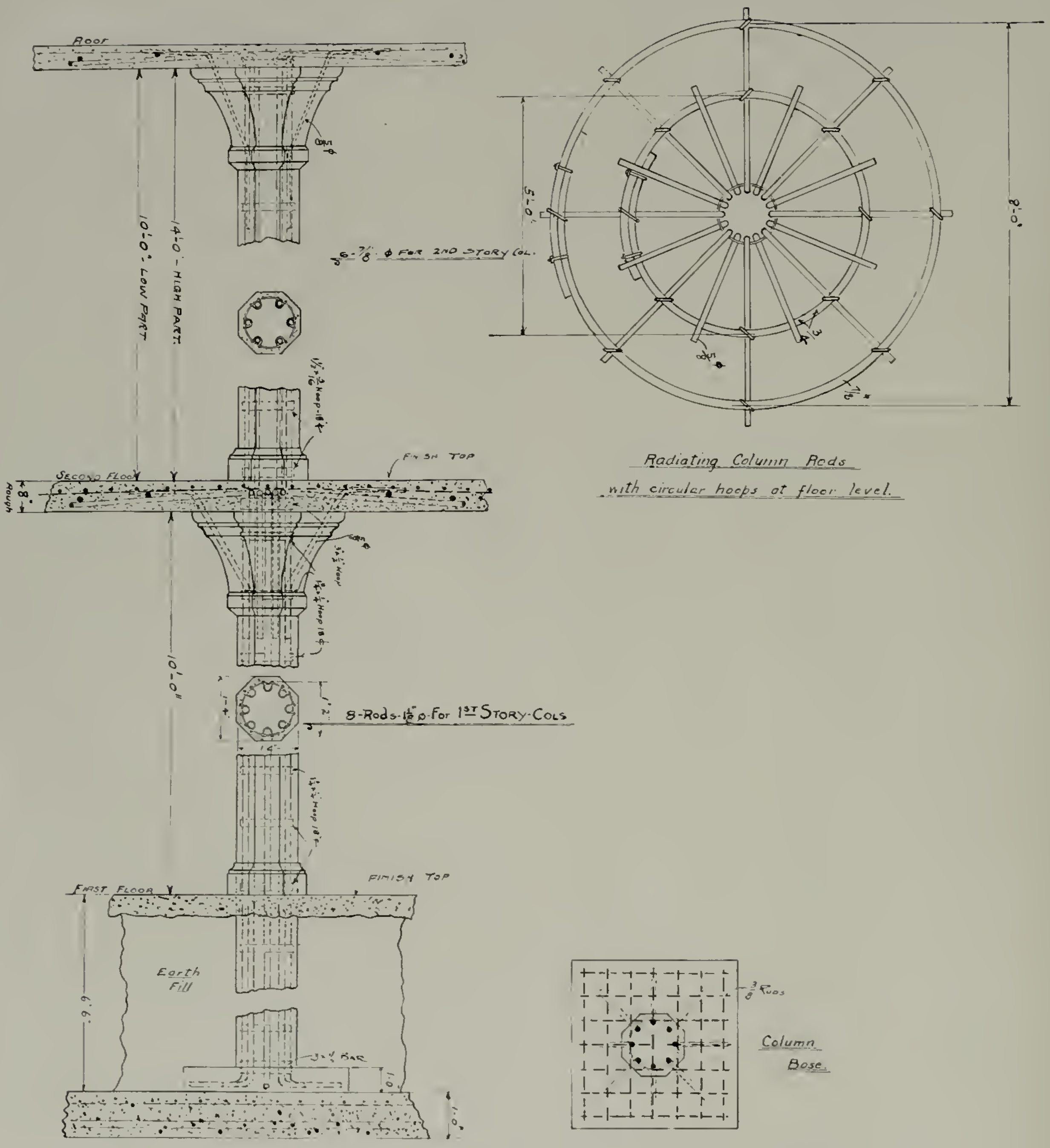
From time to time we propose to present through the columns of this journal examples of different recognized types of reinforcement, in order to bring their peculiar features more clearly to the attention of our readers. In this article we present an example of the Turner system of reinforcement, which has been used in the past few years to considerable extent in the states of Wisconsin, Minnesota, and the surrounding region.

The Turner system of reinforced concrete, so far as we are aware, has not before been used in San Francisco, and therefore the building for the Western Meat Company at Sixth and Townsend Streets, may properly be said to be of unusual design. The structure is being built by the Alling Construction Company, and is to be of reinforced concrete throughout. In plan the building measures 275 feet on Sixth Street by 137½ feet along Townsend Street. It is to be two stories high.

The foundation consists of the usual columns and wall footings, which columns and footings rest upon a mat composed of a slab of concrete 12 inches in thickness, reinforced with wire fabric. This mat covers the entire area of the building and may be said to be the unique feature of the building. In fact, one of the peculiar features of the Turner system is the use of continuous slabs, as will be noted below in connection with the remarks on the design of the floor system.

After the foundations are completed, an earth-fill 6 feet in depth is to be placed upon the 12-inch reinforced concrete mat; and upon this fill the first floor concrete surface will be laid. The second floor and roof, as already indicated, are to be of the Turner system of reinforced concrete. The accompanying illustration gives a good idea of this system.

The columns are of octagonal shape in section, and are reinforced with eight ¾-inch round rods. These rods extend upward to within about two inches of the floor line for any story, and then bend outwardly and slightly downward. The upper projection of these rods radially is about four feet. Therefore, when looking downward at the top of a column before the concrete has been placed in the floor, the radially extending rods resemble the spokes of a wheel. Upon these spokes two rings are fastened (see illustration), the outer



Drawings Illustrating the Turner System of Reinforced Concrete, to be Used in the
WESTERN MEAT COMPANY'S BUILDING

one 8 feet and the inner one 5 feet in diameter. Upon the rings are laid the reinforcing rods for the floor system, and these rods extend from column to column diagonally as well as longitudinally and transversely; thus covering the entire space in the floor plan between the columns with a network of steel bars. Eight inches of concrete is then placed upon the forms and the floor is complete.

The girders between columns and the beams ribbing the lower side of the floor slab, so usually seen in other forms of concrete construction, are absent in the Turner system. Thus, a clear ceiling height is obtained for warehouse purposes; or a very attractive ceiling, unbroken by lower flanges of girders and beams, may be obtained for office purposes.

While the Turner system is comparatively a new form of concrete construction, it has passed the experimental stage. The cities of Chicago, Minneapolis, Milwaukee and Winnipeg each have one or more buildings of this type.

These buildings have been subjected to a test load of more than two and one-half times the live and dead loads specified, the deflection in each case being well below the required limit of one seven-hundredth of the span.

Excavations are now completed for the Western Meat Company's building and the foundations are being put in place. Those interested in the study of various types of reinforcement will no doubt be interested to examine this building.

For the material from which this article has been prepared we are indebted to Mr. Franklin H. Coney, the San Francisco manager for the Alling Construction Company, which company is erecting the building.

The Economic Reinforcement of Concrete Beams

By C. E. Day*

WHERE a systematic cost record and segregation are kept and where a charge is made against concrete for all the labor and materials employed in the concrete as well as that employed in all temporary expedients, such as centering, scafolding, plant depreciation, etc., which are necessary to the placing of the concrete; likewise, where a similar charge is made against the steel reinforcement, it is possible to approximate very closely the unit costs of the concrete and steel respectively which enter into the completed beam or girder. It is evident that the ratio of these values will vary between limits depending upon numerous conditions; such as the excellency of the design as established by the conveniency and economy attending the actual construction; the method of operation and administration, the organization and equipment of plant as well as the basic cost of the raw materials and the local conditions as respects the labor market and distance from points of distribution, etc.

Obviously the ratio of the cost of the concrete to that of the steel must be known previous to the completion of the structure to be of value in the design. It is possible with wise forethought, drawing upon past expèrience, to choose representative values for a set of conditions which might reasonably be expected to prevail, which will yield a value not very far from the true one. A conservative estimate can be made only by taking into account all of the elements which enter into the completed structure and estimating how much each particular element of cost will vary with the volume of the steel or concrete..

In view of the widely different unit cost of reinforced concrete as obtained in actual practice, it naturally leads one to believe that a correspondingly different ratio of the costs of the steel and the concrete will obtain. Furthermore, in view of the great difference in the unit cost of reinforced concrete and that of plain concrete, it follows that the ratio of the cost of the steel to that of the concrete is, in general, greater than that obtained by a direct comparison of the costs of the raw materials, so that, a more or less arbitrary choice of this ratio, so based, may lead to conclusions, as regards economy, as far remote from the true conditions as the present practice of proportioning the reinforcement so that the steel and the concrete in the beam are simultaneously strained to their respective working limits.

Before entering into the deduction of the formulæ it is deemed advisable to consider the internal stress conditions in a reinforced concrete beam subjected to pure bending when different percentages of tensile reinforcement are employed.

It is well known that when a very small percentage of reinforcement is employed the greatest moment of resistance of the beam is developed when the metal is stressed to its ultimate resistance. Since the total compression in the concrete must equal the total tension in the metal reinforcement (ignoring the tension in the concrete), it follows that the concrete contributes very inefficiently to the beam action. By way of example, if the unit working values of the concrete and the steel are 500 and 16,000 lbs. per square inch respectively, and the ratio of the elasticity of the steel to that of the concrete is 15, and if 0.1 per cent reinforcement is employed, only two-fifths of the full working value of the concrete is developed. Under this condition of reinforcement, the beam may be said to be exceedingly economical in steel reinforcement;

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first, because the leverage is the greatest possible; second, because the steel is developing its full strength. If now the percentage of the reinforcement be increased, the efficiency of the concrete will be raised while that of the reinforcement will be sacrificed, but to a lesser degree. If the percentage of reinforcement continue to be increased that percentage will be reached at which the concrete and the reinforcement will be simultaneously stressed to their full strength. This percentage is often, but erroneously, called the economic reinforcement. M. Paul Christophe, in his Beton Arme, has assumed this as the economic reinforcement and studied the relative economy in the use of steel and the use of iron when the same volume of the reinforcement costs fifty times that of the concrete.

For a still greater percentage of reinforcement the greatest moment of resistance is developed when the concrete is stressed to its ultimate resistance. The reinforcement loses in efficiency in the beam for two reasons; first, the leverage is decreased; second, the unit stress is lowered.

Thus it is seen that during the addition of reinforcement, the efficiency of the concrete is gained at the expense of that of the reinforcement. That point beyond which it will not be economical to increase the efficiency of the concrete will depend upon the relative unit costs of the two materials in the completed structure.

The development which follows applies to beams reinforced on the tension side only. It is assumed that the reinforcement takes all the tension, that a plane section before flexure remains a plane, after flexure, and that the coefficient of elasticity of the concrete is constant.

The notation adopted is the following:

E=coefficient of elasticity of the concrete.

nE=coefficient of elasticity of the steel.

C=greatest intensity of bending compression in the concrete.

S=average intensity of bending tension in the steel.

b=breadth of beam.

d=distance of the extreme compression "fiber" of the concrete to the center of gravity of the steel section.

a=area of normal section of the steel.

x=distance of extreme compression "fiber" of concrete to the neutral plane.

LIST OF EQUATIONS

$$\frac{C}{x} = \frac{S}{n(d-x)} \quad \text{-----} \quad (1)$$

$$x = \frac{nCd}{S + nC} \quad \text{-----} \quad (2)$$

$$Sa = \frac{Cbx}{2} \quad \text{or,} \quad a = \frac{Cbx}{2S} \quad \text{-----} \quad (3)$$

$$M = \frac{Cbx}{2}\left(d - \frac{x}{3}\right) \quad \text{-----} \quad (4)$$

$$H = \frac{ra + db}{\frac{Cbx}{2}\left(d - \frac{x}{3}\right)} \quad \text{-----} \quad (5)$$

$$H = \frac{1}{d}\left\{\frac{r}{S}\cdot\frac{3S+3nC}{3S+2nC} + \frac{6(S+nC)^2}{nC^2(3S+2nC)}\right\} \quad \text{-----} \quad (6)$$

$$r = \frac{2S^2}{nC^2}\left\{\frac{2S^2+4nSC+n^2C^2}{3S^2+6nSC+2n^2C^2}\right\} \quad \text{-----} \quad (7)$$

$$r = \frac{4S^2}{3nC^2} \quad \text{-----} \quad (8)$$

$$S = 0.86\sqrt{nr}\text{, approximately} \quad \text{-----} \quad (9)$$

$$bd^2 = f\frac{M}{C} \quad \text{-----} \quad (10)$$

$$bd^2 = \frac{6.17 \times 1\,000\,000}{500} = 12\,340 \quad \text{-----} \quad (11)$$

$$d^3 = \frac{12\,340}{0.7} = 17\,630 \quad \text{-----} \quad (12)$$

---

r=ratio of unit cost of the steel to that of the concrete in the completed structure.

The constancy of plane section for a constant is given expression in equation (1), which readily reduces to equation (2). (See series of equations appended to this article.)

The equality of the total compressive and tensile forces is expressed by equations (3).

If the cost of the concrete per unit of volume is taken as a basis, the total cost of the beam per unit of length is ra plus bd. Taking moments about the center of gravity of the steel, the moment of resistance M at the section under consideration is easily found, and is given by equation (4).

It naturally follows that the cost H per unit of resisting for a unit of length is as expressed by the second member of equation (5).

If substitution be made in equation (5) from equation (2) and from the second one of equations (3), so as to eliminate the quantities a and x, there will at once result, when simplified, the value of H given by equation (6).

It is to be noted in equation (6) that the cost per unit moment of resistance is independent of the breadth. It varies directly with the depth and in the inverse ratio. Also, the cost depends upon the value of the stresses S and C.

D is an independent variable but S and C are dependent variables having the functional relation expressed in equations (2) and (3). The proper procedure, therefore, in actual design will be to determine the proper values S and C, and let D be determined from practical considerations.

In the analysis of the expression for the economic reinforcement two conditions arise as was pointed out above; first, when the stress in the steel is the limiting factor which is the case when a small percentage of reinforcement is employed; second, when the stress in the concrete is the limiting factor and a large percentage of reinforcement is employed. That is to say, first, S in equation (6) may be considered a constant taking, its greatest value while C assumes values varying from zero to its superior limit: second, C may be considered a constant at its greatest value, while S varies form its superior limit to zero.

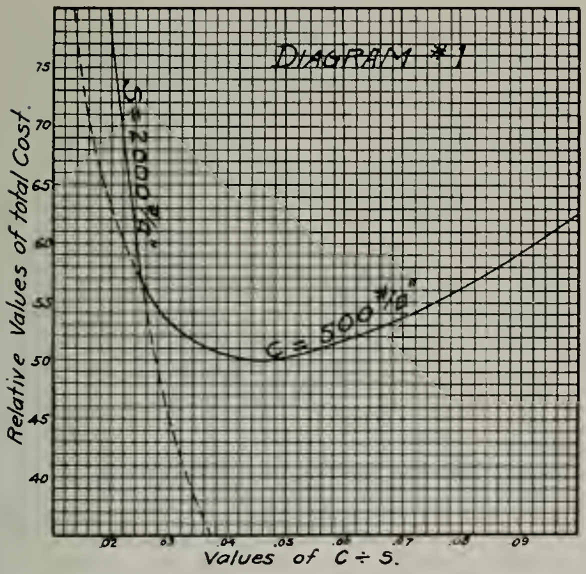


Diagram No. 1 has been constructed from formula (6) so that the abscissa of any point in the curve is the value of C divided by S and the alternate is the relative cost. Two curves have been constructed, one for S constant and equal to 20,000 lbs, per square inch; the other for C constant and equal to 500 lbs. per square inch. In both cases n=15 and r=50. The dotted portions of each curve correspond to those values of C divided by S when one or the other of the values of S and C is stressed beyond the working limits 20,000 lbs. and 500 lbs. per square inch respectively. This portion of the curves has only a theoretic value. Obviously the point of intersection of the two curves indicates the point at which each is stressed to the assigned working limits. Keeping in mind that an increase in the ratio of the stresses C divided by S implies an increase in the percentage of reinforcement and starting with the curve S=20,000 it is seen that the cost is a decrease up to the point of intersection of the two curves, and that a further economy is effected by following the curve C=500 to the point where C divided by S =0.045, which corresponds to S1= 1,000 lbs. per square inch. It is seen that it will be necessary to develop the expression for the C constant, only taking care that the ratio S divided by C does not correspond with a value of S greater than the working limit. Should this latter occur, then the maximum economy will be obtained when S and C equal their respective working values.

If the first derivative of H in equation (6) be taken with respect to the variable S and placed equal to zero, the condition for a maximum will have been satisfied. Performing the operation indicated and reducing, an equality results, as shown by equation (7).

That is to say, the maximum economy will have been obtained from a given working value of C when such a percentage of reinforcement has been introduced that the steel takes that value of the stress S which renders the expression on the right of equation (7) equal to the ratio of the cost of the steel to the cost of the concrete. This expression does not admit of direct solution; and, the solution might be considered tedious.

It is to be noted that if the coefficient of $n^2 C^2$ in the numerator of the parenthesis of equation (7) were four-thirds instead of unity, the parenthesis would reduce to $\frac{2}{3}$, in which case the equation would take the form of equation (8) or (9).

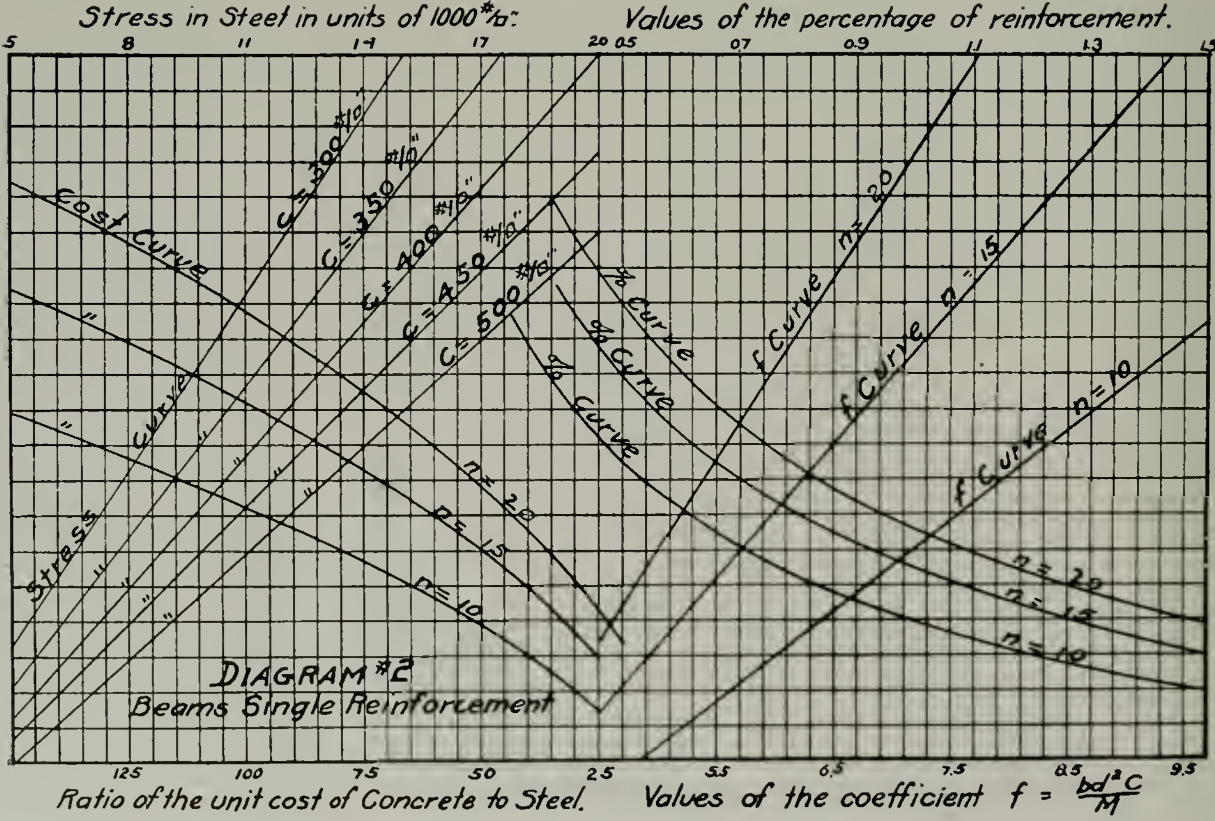
Diagram No. 2 is based on equations (2), (3) and (7). The diagram has been constructed to suggest how readily the main proportion of a beam or girder may be determined and thereby forestall any argument which might be urged relating to its complexity. The vertical projections of the intersection of the various curves with any horizontal line indicate corresponding values of the stress in the steel, the ratio of the unit cost of theel to concrete, the percentage of reinforcement and values of the coefficient f. f has the significance indicated in equation (10), where b is the breadth, d the depth from the most compressed fiber of the concrete to the center of the reinforcement, M is the external bending moment and C is the unit stress in the concrete at the extreme compressive fiber. By way of illustration, suppose it is desirable to design the main proportions of a beam to sustain a maximum bending moment of 1,000,000 inch-lbs. when the working value of the stress in the concrete is 500 lbs. per square inch, the ratio n of the coefficient of elasticity of the steel to that of the concrete is 15, and when the ratio of the unit cost of the steel to that of the concrete is 70. Enter the diagram where the ratio of the cost of the steel to that of the concrete is 70. Follow vertically to the intersection with the cost curve n=15. The intersections of a horizontal line through this point with the other curves indicate that S=12,700 lbs. per square inch, that 0.722 per cent of reinforcement is required and that f=6.17 From equation (10) we have, by substitution, equation (11). If it is considered desirable to have b=0.7d then, equation (12) results from equation (11), and d=26 inches, which value may be taken from almost any hand book. It follows that the breadth is 18¼ inches and the reinforcement is 3.42 square inches. Other uses of the diagram will suggest themselves.

The diagram shows clearly the remarkably different results which might be obtained from the choice of the constants n and C in the equations. Also, the diagram lends emphasis to the fact that where any pretense of choosing the economic percentage of reinforcement is made, a careful estimate of all those elements of cost entering into the completed structure, which vary with the volume of the concrete and steel, is necessary.

In closing, the writer regrets that he has no data of costs to present and suggests that a discussion on the relative costs of steel and concrete in reinforced concrete constructions would be at once interesting and instructive.

NOTE.—Prof. Mansfield Merriman in a recent Transaction of the Am. Soc. C. E. has made a similar study of this subject. In order that the writer may not be wrongly judged, he wishes to say that the essential features of this paper had been developed previous to any knowledge of the above reference.

Berkeley, December 30, 1907.



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SUBSTANTIAL PIERS FOR SAN FRANCISCO

IN our last issue we presented to our readers two articles on "Advanced Pier Construction in San Francisco," one article by Mr. Carl Uhlig, and the other by Mr. F. A. Koetitz. Both of these gentlemen, by their experience, are extremely competent to speak for this type of construction. In their articles it will be noted that reference is made to the work of Mr. Howard C. Holmes, for some time chief engineer to the Dock Department of San Francisco. Mr. Holmes has virtually been a pioneer in concrete dock construction, and his excellent article upon this subject in the Encyclopedia Americana gives a clear idea of the merits of docks constructed from his plans.

San Francisco has had severe problems to solve in the construction of docks, because of the extreme destructive activity of the wood-eating animals in California waters. It is not surprising, therefore, to find on this coast probably the most advanced types of dock construction in the country.

In our judgment the best type of dock embodies the following features taken from the works of the three gentlemen named above: (1) wood-piling encased in concrete above the permanent low-water line; this concrete put in place by means of a wood coffer-dam, the same concrete reinforced when necessary; (2) a dock floor of structural steel beams anchored securely to the top of the piling described under heading (1); (3) a reinforced concrete floor built upon the structural steel frame of No. 2, designed not merely for strength, but for sound protection to the rolled steel floor system, in order to decrease to a minimum the tendency to corrosion; (4), fender piles around the dock line protected by springs.

We do not believe in a floor system for a dock composed entirely of reinforced concrete. There is entirely too much impact possible upon such floors. We believe that the frame of the floor should preferably be of rolled steel, although it is entirely possible to use girders of reinforced concrete to support the slab.

We hope that dock construction of this type will become more and more fashionable in San Francisco, and that increased first cost will be accepted as an assurance of eventual economy. There is no reason why San Francisco should not have the best docks in the country, just as there are many reasons why it is fatal to use in our waters an ordinary wood construction.

TECHNICAL LITERATURE

A NEW monthly journal has just been started in New York with the name of "Technical Literature." The first number appeared for the month of January, 1907. As its name implies, the journal is to be a monthly review of current technical publications, and is to be devoted to "the interests of all branches of the engineering profession to a greater or less degree." It is proposed that the journal shall include a review and index of the more important articles, with comment, appearing in the various high class technical periodicals; also reviews and announcements of new technical books; notes from technical societies and colleges; original articles and information on all subjects that can be classed under the head of technical literature.

A paper with this purpose in view can satisfy a great need at the present time. Busy engineers and practitioners and equally busy students and teachers of engineering subjects find it utterly impossible to keep abreast with current technical publications; first, for want of time; second, because of the expense that would be entailed by subscribing to even a portion of the technical weeklies and monthlies now on the market.

That this need has been felt keenly for some time is evident by steps which have been taken by various engineering interests for the purpose of indexing current publications, and especially pamphlets and important articles occurring in periodicals; thus, for example, for some time past, the American Society of Civil Engineers, in its Proceedings, has given considerable attention to this need under the heading "A Monthly List of Recent Engineering Articles of Interest." The Engineering Magazine also has for some years been giving a similar list, somewhat wider in extent than that of the American Society of Civil Engineers. The same publishing company, namely, The Engineering Magazine, has just placed before the public its new Engineering Index, 1901–5 inclusive, including 43,000 descriptive notes. The volume is an alphabetical index by subjects to the technical literature of the entire world, from January, 1901, to December, 1905, inclusive, and is complete in itself for these five years.

We might name further examples which would indicate, on the one hand, a recognition of a long-felt want, and, on the other, a desire to satisfy the necessity that current engineering literature be classified and indexed.

One reason for these necessities arises from the fact that a great mass of second-class matter is regularly published, so that the first-class publications and pioneer works are buried in a mass of print.

We are glad to welcome Technical Literature to an already long list of weekly and monthly magazines, despite the fact that it is to be deplored that so many new magazines are constantly being born, because we believe that the purpose of "Technical Literature" is timely; and moreover, because we believe in its ability to meet successfully its problems and its requirements, on account of the resourceful backing and experience of its promoters.

Building For the Boyd Estate

California and Drumm Streets

IN appearance this building will be of simple commercial type, fitted for offices for shipping and marine agencies. The basement will be used for a restaurant and barber shop.

All floors are to be provided with steam heat; an exhaust fan will be used for the ventilation of the restaurant.

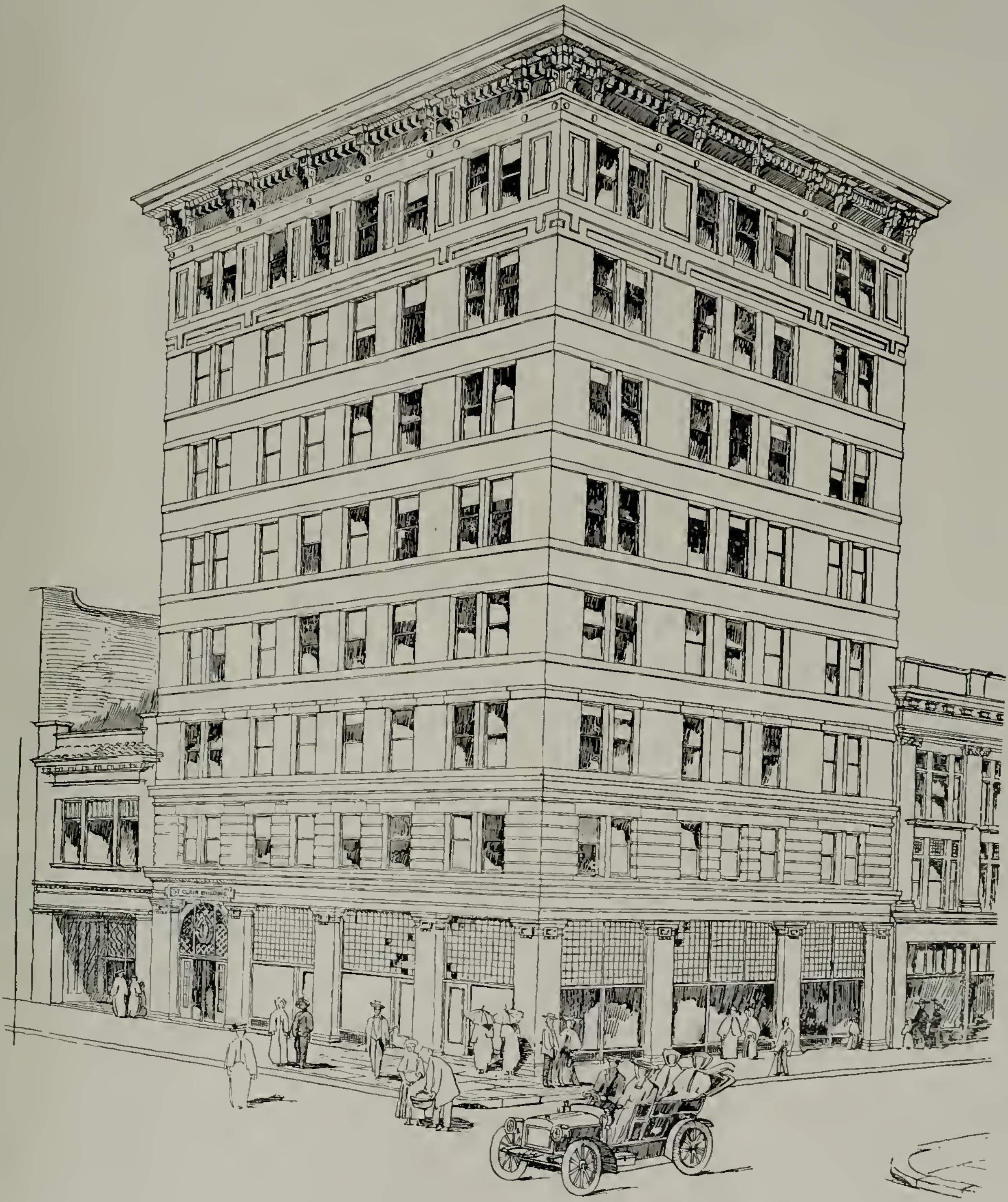
On the Drumm Street side the building will be equipped with fire escapes. A stand-pipe will run to the roof and there will be hose-reels in the corridors. In the construction of the building woodwork will be entirely eliminated. The window frames are to be hollow metal, with reversible double-hung sash, glazed with wired plate glass. The interior finish and doors will be of metal. Terra cotta will be employed for office partitions. All walls and ceilings will be furred and plastered on metal lath. The floors will be of cement finish.

A large light-well in the rear will give an abundance of light and air in all offices. Two 500-gallon water tanks will maintain the water service throughout the building.

The building is to be equipped with two quick running direct connected electric elevators and also with two sidewalk elevators.

Structurally the building is of reinforced concrete throughout. To obtain a massive appearance, the two street elevations are being made seventeen inches thick, thus allowing the windows to be set well back from the faces of the walls. The wall spaces between columns and window jambs are being boxed out, so as to leave a 6-inch face wall, thereby reducing the amount of concrete, and thus the dead load of the building. Cornices and projections are all moulded and poured into place. The foundation is of piles, capped with reinforced concrete footings, from 30 inches to 36 inches thick. Reinforced concrete struts run between and separate all footings, thus stiffening them in place.

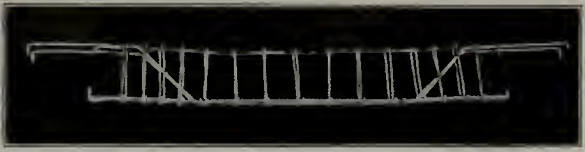
All columns are reinforced vertically with rods anchored in the footings and connected at each story with threaded couplings, thus making them continuous from bottom to top of building; these rods are designed to take



N. Blaisdell, Architect

S. A. Jubb, Engineer for Owners

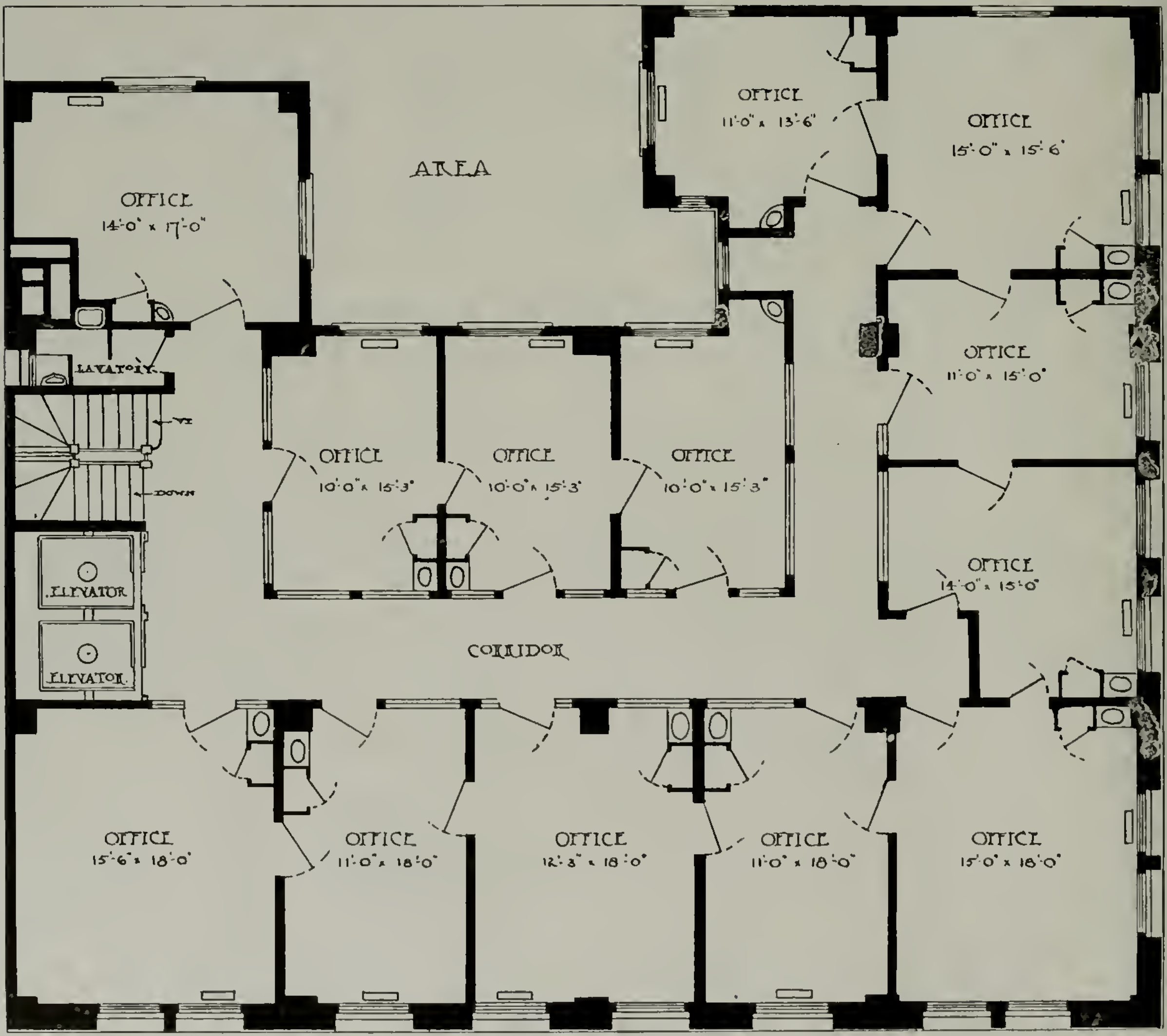
REINFORCED CONCRETE BUILDING, FOR THE BOYD ESTATE
At California and Drumm Streets, San Francisco.



UNIT TRUSS
Used in the Boyd Estate Building.

eccentric and wind stresses only. The interior columns were reduced in size by the use of a spiral winding of high carbon steel wire. All beams and girders are reinforced with the Unit Girder type of fabricated reinforcement, consisting of round, medium steel rods varying from ¾ to 1⅛ inches in diameter, with inverted U-shape stirrups for shear members, all being connected in such a manner as not to become displaced while handling or in placing the concrete. The trusses are made up and brought to the building and placed directly in the forms.

By the use of this Unit Truss all the reinforcement on a floor is placed in two days. It can readily be seen that all loose pieces left to be handled in the building are reduced to the minimum, the continuity and shear members being a part of the fabricated truss. This method assures the accurate placing of all parts of their predetermined positions, which is the only salvation of a combined material like reinforced concrete. Ample web members or stirrups are used and of a type which is believed to be the most efficient, bonding, as they do, the slab to the beam, also increasing the compression value of the concrete very materially by acting in a manner similar to a looping of the concrete and with only a fraction of the steel required when longitudinal rods are used, as many tests have proved. The floor slabs and walls are reinforced with plain



BOYD ESTATE BUILDING
California and Drumm Streets, San Francisco.

rods. All reinforcement is held away from the forms with concrete blocks. A special collapsing floor form is used which will be shifted from floor to floor.

For the information given above, we are principally indebted to Mr. S. A. Jubb. Mr. Nathaniel Blaisdell is the architect, and the Frank B. Gilbreth Company are the contractors. The accompanying drawings give additional features of this building, which we believe to be one of the most satisfactorily designed and soundly built reinforced concrete buildings now being erected in San Francisco.

Some Observations on the Lateral Vibrations of Tall Buildings

By F. E. Schmitt, Jr. *

THE intense interest in earthquake construction which has been created by the necessities following the San Francisco disaster of April, 1906, demands an answer to the question how our tall steel-frame buildings behave in the matter of lateral vibratory resistance, and to the subsequent question how their resistance may be developed to the degree called for by earthquakes. Full and complete answers to these questions can hardly be expected before much time and study has been devoted to them. In the meantime any available items of information bearing on them should be brought together. In this view the writer presents the following matter on the lateral vibrations of tall buildings.

Late in 1903 the writer became interested in learning the kind and amount of lateral motions of typical modern tall buildings under the influence of wind. Study of the question led to the design of apparatus for recording these motions, and in the following year the apparatus was constructed in accordance with this design. The design has proved to be well suited to the needs of the case, and while some crudities have proved annoying, yet the apparatus has performed sufficiently well in actual use.

Observations, more or less extensive, were made on three tall buildings, and the records obtained were carefully studied. It is recognized that three cases are far too small a number to give results of very general validity, but time and opportunity have been lacking hitherto for further work. Nevertheless, since the records obtained in the three cases mentioned are quite consistent, their results are given in the following, in so far as they pertain to the present subject.

The apparatus itself need not be described in detail here. Essentially it consists of (1) a sensitive element, (2) motor-driven recording gear which draws a continuous paper strip over a record cylinder, and (3) two pen-levers extending from the sensitive element to the record cylinder.

The sensitive element is an inverted pendulum, consisting of a heavy weight fastened to the upper end of a light steel rod which is fixed vertically in a base socket. It was designed to be of such proportions as to have a proper period of about 10 sec. Actually it can be adjusted for periods lying between the approximate limits, 3 sec. and 11 sec. It has been found best to work with the highest practicable period. Most of the records taken were made with periods approximating 7 sec. Larger periods generally involved so narrow a zone of stability that continuous working was impossible.

The two pen-levers are connected to the sensitive elements by links at right angles to each other, so that they will give, for instance, the N-S. and E-W components of motion. The pens are glass tubes drawn down to a fine opening whose edges are rounded in the flame. Solutions of aniline colors are used as ink. A clock fitted with contacts to make an electric circuit once every minute is connected with a third (magnet-operated) pen, giving a time-scale along the moving strip of paper, and a push-button shunted around the clock provides for recording the instant of occurrence of any special phenomenon it is desired to note. The whole is mounted on a base-plate resting on three leveling-screws.

For use, the apparatus is set on the floor whose motions are to be registered, and is leveled up to bring the sensitive element into

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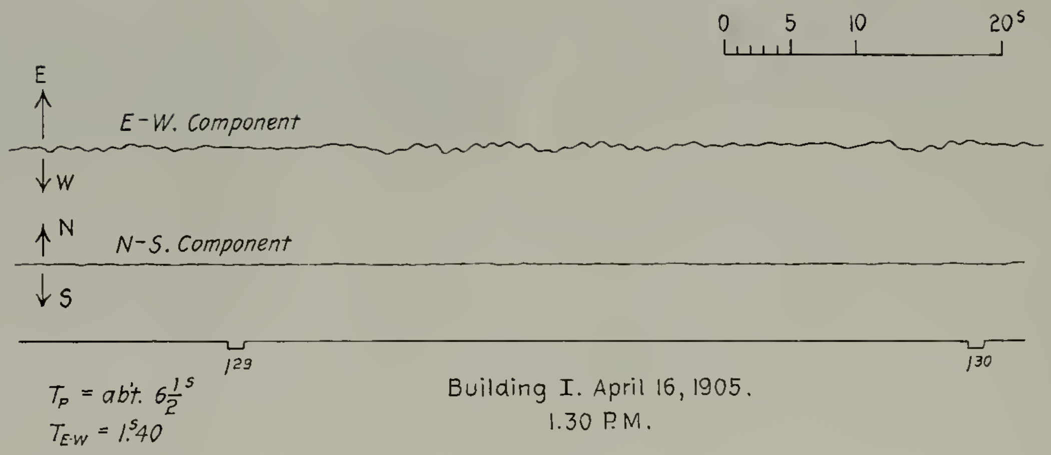
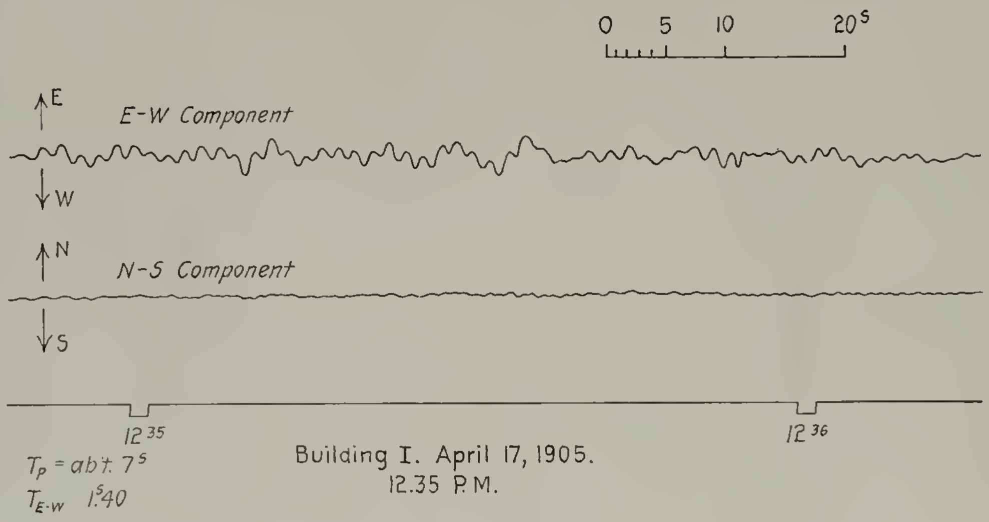
*Associate Editor, "Engineering News," New York City.



A OF FIGURE 1



B OF FIGURE 1

central position; after the latter is adjusted for period, the motor and clock circuits are closed. Walking about near the instrument must be avoided, as with all but the most rigid floors the deflections produced are sufficient to introduce erratic irregularities into the record.

The principle of action of the apparatus is very simple. Suppose the floor on which it stands executes horizontal vibrations of simple harmonic type. The sensitive element will be thrown into vibrations, of character determined partly by the proper period of the element and partly by the period of vibration of the floor. To avoid resonance (and consequent meaningless records), the proper period of the element must differ considerably from that of the floor, and to secure the desired results it must be several times larger than the latter. That is, if the motion of the floor has a period of say 1 sec., the natural period of the element should be at least 4 or 5 sec. Under these circumstances, the period of motion of the sensitive element in space will be almost wholly determined by its natural period of vibration, and only to a very small degree by the period of the floor's motion. Now, since the strip of paper on the record cylinder, being connected firmly with the frame of the apparatus executes the same vibrations as the floor, the movement of the pens over the record strip will be the sum or difference of the absolute motions of the sensitive elements and of the floor. The record will therefore show two superimposed vibrations, one that of the sensitive element, the other that of the floor; the former being of much longer period, the record will show the shorter period floor vibration superimposed on the longer wave-line of the sensitive element's own motion.

This relation can de developed mathematically without difficulty, but will be evident enough without this. It must only be borne in mind that any approach to equality between the two periods in question gives rise to interference of increasing amount, and finally, at coincidence of the two vibrations, to complete resonance checked only by frictional damping. Such records will be misleading. The records shown farther on do not, however, involve such conditions, as the natural period of the sensitive element was always made sufficiently large.

If the motion of the floor be not a simple harmonic vibration, but a vibration of other type, or an irregular periodic motion, it may be considered as equivalent to the sum of an infinite series of harmonic vibrations, of different periods and amplitudes. The remarks above apply to each component vibration, provided its period be not large enough to produce an approximation to resonance with the instrumental period. Therefore, the same principles apply to the sum of the series, that is to the irregular vibration in question.

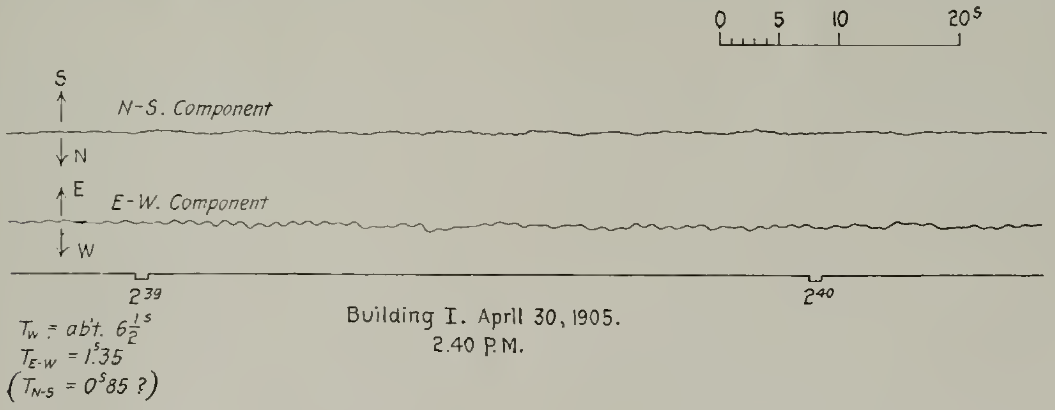
These considerations lead to the expectation that a properly designed instrument of the type described will produce records from which the form and amount of the vibrations of buildings may be learned.

Portions of six record strips from the three buildings, in which experimental runs were made, are placed in evidence in Figs. 1, 2 and 3 herewith. Each specimen covers a period of a little over one minute, as the time-scale indicates, and is typical of the records taken in that building. No vertical scale is given, as any reference to the amplitude of the motion would only distract attention from the points which it is desired to exhibit.

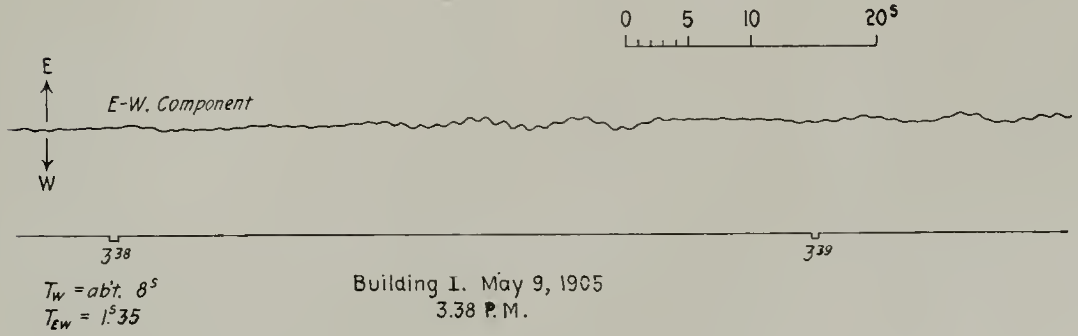
In the observation runs represented by these sample strips, the sensitive element of the instrument had a period (Tp) of 6 to 7 sec. The internal damping is relatively powerful for motions of such long period, and in consequence they have left little trace in most of the records. It happens that all four of the specimens in Fig. 1 show rather clearly the proper vibration of the sensitive element, which fact is due to the rather rapidly varying nature of the external motions in these cases.

Prominent in all the specimens shown is a wave of short period, of more or less variable amplitude, but of great regularity as concerns adherence to smooth wave-motion. Measurement of the record strips* shows that this persistent wave is of constant period for a given building. It is the lateral vibration of the building itself.

The vibration is persistent, in the sense that



C OF FIGURE 1



D OF FIGURE 1

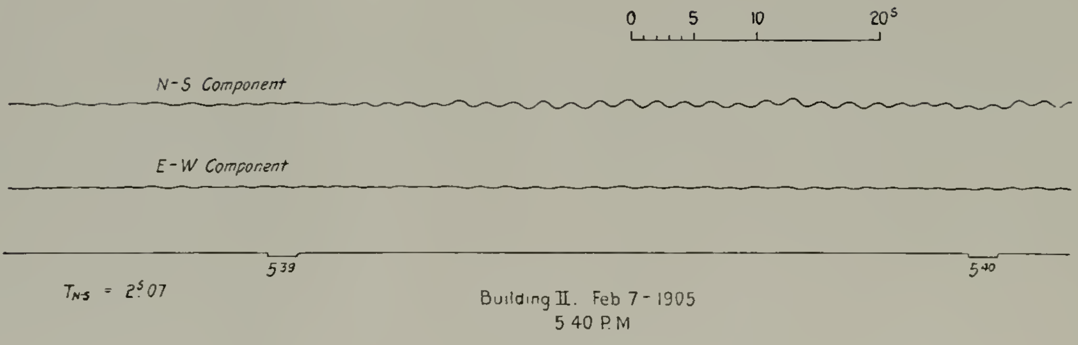


FIGURE 2

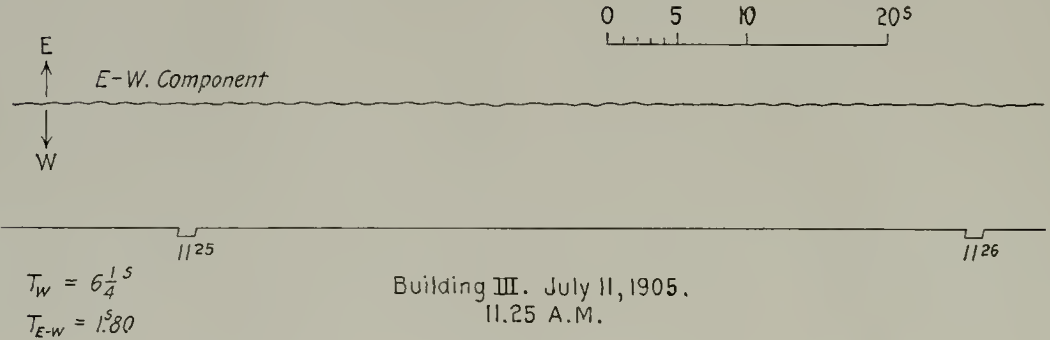


FIGURE 3

it is always present, provided a fair amount of wind is blowing. The instrument is sufficiently sensitive to record the vibration even when its amplitude is less than 0.01 in., so small, in fact, that the presence of a wave in the line is detected only by closest scrutiny. A cylindrical spectacle-lens, held so as to magnify the vertical scale only, proved useful in going over the strips.

The records show, beside the short-period vibration in question, occasional irregularities, prominent in the four samples of Fig. 1, and particularly so in sample C, where the irregularities in the pen-trace are more apparent than the short regular vibrations. These irregular motions are to be disregarded in the present examination. Suffice it to say that they are not due to the instrument but undoubtedly originate in the building. They may be thought of as being chargeable to the pulsations of the wind pressure, and consisting of irregular oscillatory motions of variable but certainly much longer periods than the short vibration. As already stated, such irregular motions can (in theory) be resolved into a sum of regular harmonic motions (certain of which components would, in the present case, be affected by large logarithmic coefficients representing damping, etc.), and their shorter-period components are recorded by the instrument.

To illustrate the fact that wind pulsations do occur of sufficient amount and irregularity to account for such long-period irregular vibrations of a building, two wind-pressure diagrams are shown in Fig. 4. These were traced by the same recording instrument as produced the vibration records Figs. 1 to 3, but by a separate pen-lever, actuated by a pressure-gauge of novel construction. The gauge, which proved very satisfactory in sensitiveness, rapidity of response, portability, and consistency, though altogether home-made, may for the present purpose be dismissed with the remark that it is a modification of a form of pressure-gauge described by Prof. William Kent.* It is called by him the "tomato-can" gauge, but is meritorious enough to deserve a better name; the writer prefers to call it the Kent gauge.

The indications of this page represent static or barometric pressure, not dynamic effect of moving air. The receiving tube, passed through the opening of a window to the outside of a building,† received at its outer end the static pressure of the outer air. The gauge measures the difference between this pressure and the pressure in the room. It is assumed that the interior of the building has a barometric pressure equal to that of still (or undisturbed moving) air in the open. Then when the receiving tube is out on the windward side of a building, the gauge measures the air pressure against the building due to the piling up of air on that side; when out on the leeward side, it measures the vacuum there existing due to the drawing away of air by the deflected air currents. Thus, specimen A of Fig. 4 shows the pressure on the windward face of a building, by ordinates measured upward from the time-line. In specimen B of Fig. 4, the ordinates measured downward from the auxiliary line, which was repeatedly verified as the zero-line, show the (negative) pressure on the flank of the same building, near one of the windward corners, on the same day. The ordinates of the original are not very far from equivalent to water-heights; the vertical scale is given in the figure in terms of pounds per square foot.

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*The time clock used had the peculiar defect that the second-hand drive, from which the contacts were taken, makes one revolution in about 56.2 sec. instead of in 60 sec., although the minute hand kept good time. A time-scale has therefore been placed on each record-slip.

*Appendix to Report of Boiler Trials Committee, Trans. Am. Soc. M. E., Vol. 71 (1900), p. 89

The time-scale of Fig. 4 is very much more condensed than that of Figs. 1 to 3. Bearing in mind this difference, it will be seen that the wind pulsations are of rather long period when compared with the short-period vibrations in Figs. 1 to 3, but are of suitable interval, amount and suddenness to account for the possibility of the irregular motions which have been referred to. It should be evident, at least, that the irregular motions in the vibration records have no direct connection with the short-period vibrations.

It is thought, also, that the pressure records in Fig. 4 show conclusively that the short-period vibrations in Figs. 1 and 3 are not to be viewed as inelastic swaying in tune with the variations of wind-pressure; for they are much more rapid and more regular than the variations in wind-pressure. (Fig. 4 is typical of the large number of wind-pressure records made, in point of smoothness, etc.) This gives final proof, if such be needed, of the fact that the short-period vibration in the vibration records is determined by the characteristics of the building alone, and depends only for its amplitude on external causes.

Two features of this short-period vibration concern the present subject: First, its regularity and its excellent smooth wave form; second, its period.

First, the motion is regular, and the wave is neither peaked nor flat-topped, but as nearly as can be seen of sine-form. A building of loose structure, with appreciable amount of inelastic motion, moving to and fro under lateral impulses, would not give such a vibration wave, nor will a solid body rocking on a flat base do so. An elastic body fixed vertically in a socket and vibrating laterally produces just such a wave. The sensitiveness of the instrument, already mentioned, is such that the presence of any appreciable degree of inelastic or loose-jointed motion in the building would doubtless be revealed in some parts of a long observation-run at least. But the records taken, which cover many hours' running in winds of various grades, do not show such indications. On the contrary, in the strongest winds the trace of the pen is most regular in its adherence to the sine-wave form of the short-period vibrations (for example, in specimen B of Fig. 1). Moreover, anything but a highly elastic vibration would be so rapidly damped out that the persistence of the wave-motion for several periods would be virtually impossible. But all the records show that the short-period vibration is strongly persistent, at small as well as at large amplitude.

It is concluded that the three buildings represented by the records Figs. 1 to 3 are sensibly equivalent to elastic bars fixed at the base and vibrating laterally under the influence of transverse forces; and further, that the elasticity of these structures is quite perfect, as evidenced by the persistence of the vibration.

This conclusion, while it merely expresses what the writer previously thought was a **probable** fact, had, so far as he is aware, had no previous experimental basis of proof. It is a conclusion of especial importance in connection with questions concerning the effect of forced lateral distortions on the strength of buildings.

The other point to which attention is drawn is the period of the vibration shown in the record strips. It has already been stated that this period is constant for a given building, at least over the range of the observations yet made with the apparatus described. This period is, approximately:

For Building (Fig. 1): 1.4 sec. E-W.
For Building II (Fig. 2): 2.1 sec. N-S.
For Building III (Fig. 3): 1.8 sec. E-W.

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†The opening was sealed by another rubber tube as packing, so as to prevent any material inward or outward blast of air at the point of measurement.

The directions noted are in each case those of the least dimensions of the buildings. In C, Fig. 1, the N-S line shows traces of a vibration of period 0.85 sec., which, when compared with the E-W period, fairly corresponds to the relative dimensions of the building in this direction.

These values are to be compared with what may be calculated from the dimensions of the respective buildings.

A vertical prism or bar of uniform cross-section, of elastic material, fixed firmly at its lower end, may be thrown into flexural vibrations by applying a lateral force and releasing, or by other suitable means. The vibrations, considered as horizontal oscillations of a given point of the bar, will be simple harmonic motions and their period will be constant, being determined by the material and dimensions of the bar and by no other factor. The curve assumed by the bar (at any instant) may be of various shapes, depending on whether the bar vibrates as a whole, or in two or more parts, or in a motion compounded of several of these primary types. When the bar vibrates as a whole, the curve is as shown at 1 in Fig. 5; when it vibrates in two parts, its curve is as shown at 2, Fig. 5. These two curves have been carefully plotted from deflection ordinates calculated from the applicable formula; they were calculated for equal deflections at the upper end, merely in order to show the shape of the curve.

The formula for the deflection of the bar in such vibration is

$$y = \frac{M_0 l^2}{2k^2EI}\cos\left(\frac{K^2}{l^2}\sqrt{\frac{EIg}{w}}\;t\right)\left[\cosh\left(K\frac{x}{l}\right)-\cos\left(K\frac{x}{l}\right)-V\left\{\sinh\left(K\frac{x}{l}\right)-\sin\left(K\frac{x}{l}\right)\right\}\right]$$

Where y is deflection from vertical of any given point of the bar.
x " distance of given point from lower end of bar.
Mo" bending moment at base, at instant of extreme excursion.
l " length of bar.
E " modulus of elasticity of the material of the bar.
I " moment of inertia of cross-section of bar.
w " weight of bar per unit of length.
g " acceleration of gravity.
t " length of time from instant of extreme excursion to instant considered.

$\left.\begin{array}{l}\text{Cosh } K\frac{x}{l}\\ \text{Sinh } K\frac{x}{l}\end{array}\right\}$ is $\left\{\right.$ hyperbolic cosine and sine of $K\frac{x}{l}$

which in the absence of a table may be computed from:

$$\cosh(z) = \frac{1}{2}(e^{z}+e^{-z})$$

$$\sinh(z) = \frac{1}{2}(e^{z}-e^{-z})$$

k is a constant depending on whether the bar vibrates as a whole, or in two, three, etc., parts.

When bar vibrates as a whole, k is 1.8752
When bar vibrates in two parts, k " 4.6941
When bar vibrates in three parts, k " 7.8548
etc.

thereafter approximating very closely to the radian measure of $\frac{7}{2}$p, $\frac{9}{2}$p, etc., where p is 3.1416, and V is a constant derived from k, of value

$$V = \frac{\sin k}{\pm 1 - \cos k}$$

the first figure of the denominator taking the sign of sin k.

What concerns us in this formula is the period of vibration. If T is the period, then

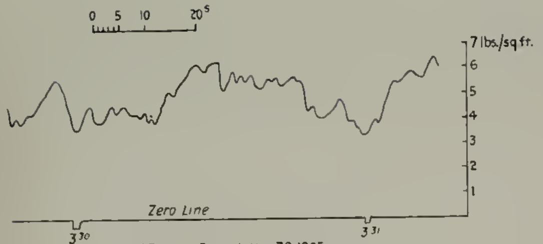
$$\frac{K^2}{l^2}\sqrt{\frac{EIg}{w}}\cdot T = 2\pi$$

$$T = \frac{2\pi l^2}{K^2}\sqrt{\frac{w}{EIg}}$$

It will be observed that the period depends on the value of k, that is, upon whether the bar vibrates in one, two or more parts. As a matter of fact, in any given case of vibration of a uniform bar, there will be present not only the fundamental but also overtones, that is, the vibration of the bar will be represented by an equation such as

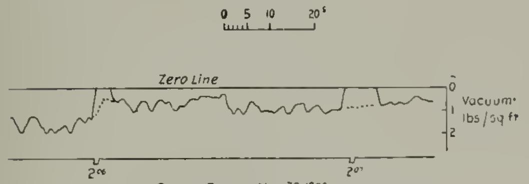
y equals Ay_1 plus By_2 plus Cy3 plus ....

where y1 has the first value of k, y_2 the second, etc. (and the periods of the several components differ correspondingly), and A, B, C are frac-



Wind Pressure Record, Nov 30, 1905
West Window, about 230 Ft above Street
Brisk West Wind

A OF FIGURE 4



Wind Pressure Record, Nov 30, 1905
South Window, about 230 Ft above Street
Brisk West-Northwest Wind

B OF FIGURE 4

tional coefficients depending on the initial curve to which the bar was sprung before releasing it.

In Fig. 5 there has also been plotted (broken line 3) the curve of deflection of a uniform bar under uniformly distributed lateral loading, and (curve 4) the curve of a uniform bar under a horizontal force concentrated at the end, each for an end deflection equal to that of curves 1 and 2 (No for equal base moments Mo). By comparing curve 1 with 3 or 4 it will be seen that a bar originally deflected by a uniform loading, or by a load at the end, will give a number of very faint overtones besides the fundamental vibration. But a comparison of curves 1 and 2 will readily suggest that a vibration with one intermediate node (i. e., in two parts) absorbs very much more energy than the fundamental vibration, and vibrations of more than one node still more so. It is to be expected, therefore, that all overtones will be subject to much more rapid damping than the fundamental, and, if originally faint, will after a few periods be vanishingly small compared with the fundamental vibration. If the tall building can be considered as such a vibrating bar, it is not probable, therefore, that in ordinary vibrations due to wind any recognizable trace will be found of other than the fundamental vibration. This has proved to be the case in the vibration records of which samples have been given above.

An important difference between the tall building and a uniform bar is that the former has variable weight (w) per unit length, and variable cross-section. It happens, however, that the equation above given applies not only to a bar of constant w and constant I, but also to one in which the quotient $\frac{W}{I}$ is constant. The differential equation from which the formula for y was found is

$$\frac{w}{g} \cdot \frac{\partial^2 y}{\partial t^2} = -EI\,\frac{\partial^4 y}{\partial x^4}$$

or

$$-\frac{w}{EIg} = \frac{\dfrac{\partial^4 y}{\partial x^4}}{\dfrac{\partial^2 y}{\partial t^2}}$$

which takes account only of $\frac{W}{I}$, not of w and I separately. The constancy of $\frac{W}{I}$ is approximately a fact in tall buildings, at least those of relatively small horizontal dimensions. The weight of the walls, which forms a large part of the total dead-weight, decreases as their thickness, and the moment of inertia of the cross-section of the building decreases in the same ratio. Often, also, the lower floors of a building are thicker and heavier than the upper ones. Actually, an approximate calculation of w and I made for the successive stories of a particular twenty-story building of relatively small ground area showed that the estimated weight per story decreased from the bottom up in only slightly lower ratio than the moment of inertia of the cross-section. For these cases, then, the formula above given may be assumed to be approximately applicable.

The equations for deflection and period, above given, involve a neglect of all shear deformation. It is fairly certain that in a tall building, when considered as a vibrating bar, the shear deformations are of strong influence. They would operate to increase the period of vibration, as well as alter the curve of deflection. In buildings having insufficient bracing in the steel frame, or bracing of the portal type only, and lacking transverse walls, a larger effect of the shear deformations upon the period of vibration is to be looked for than in buildings rigidly braced in the frame or by walls.

When the periods of transverse vibration of the three buildings in question had been obtained

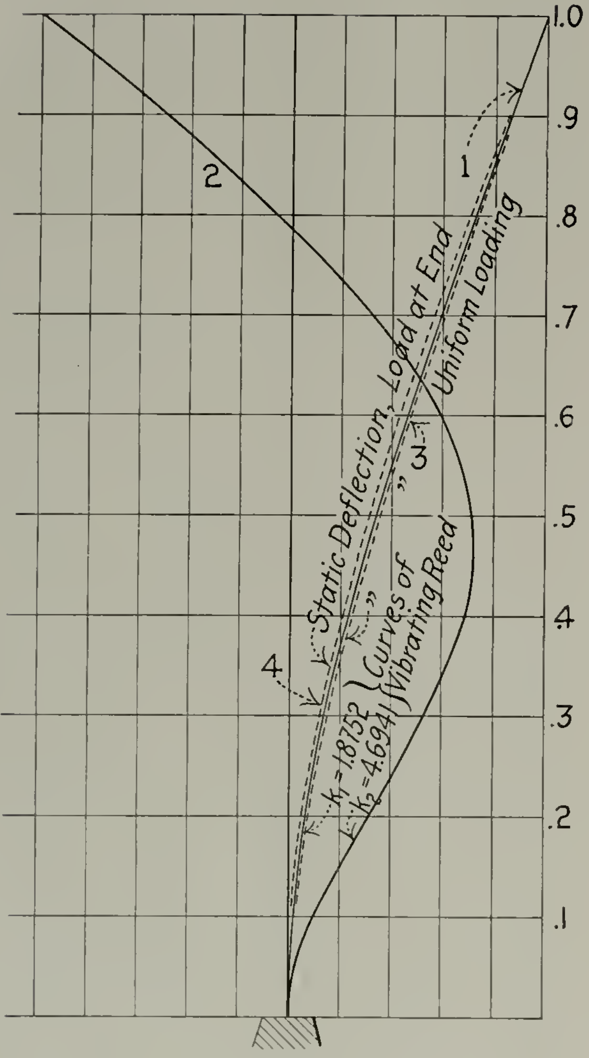


FIGURE 5

from the curves traced by the recorder, the above given formula for the period of a bar was applied to them, with the purpose of seeing in how far the behavior of the building was like that of a vibrating bar. The results a.e as follows:

Building I. An unusually small building, 18 stories high; about 23 x 74 ft. in ground plan; height, from curb to roof over 18th story, 235 ft. The length of the building is north and south. Standing on the northwest corner of a street intersection, its east face is free, and the west face is exposed above the first half dozen stories. For the larger part of its height the following values hold:

w is 75,000 lbs. per ft.; I for walls, 38,930 ft.4, for columns, 445 ft.4, both taken around a N-S axis. If E for the walls is assumed at 2,000,000 lbs per sq. in.; for the columns at 30,000,000 lbs. per sq. in.; then

for walls, EI is 112,000 x 10^8 lbs. x ft.^{2}
for columns, EI is 19,200 x 10^8 lbs. x ft.^{2}

Total, EI is 131,200 x 10^8 lbs. x ft.^{2}

$$\frac{w}{EI} = 0.573 \times 10^{-8} ft^{-2}$$

$$T = \frac{6.28 \times 2.35^2}{10^4 \times 1.875^2}\sqrt{\frac{0.573}{32.2}} = 1.31 \text{ sec.}$$

which is to be compared with 1.35 to 1.40 measured from the records. If the modulus of elasticity for masonry were taken at 1,500,000 lbs. per sq. in., the result would be T=1.48 sec. The writer found little experimental information on the modulus of elasticity of brickwork, and has assumed the above merely as probable values.

It may be noted that the calculation carried out in the same way for an E-W neutral axis, gives, with

Em—2,000,000 lbs. per sq. in., T—0.64 sec.; with

Em—1,500,000 lbs. per sq. in., T—=0.72 sec.

The building has a solid wall on its north side, while the other three walls are broken by windows. There are no cross-walls, and the partitions are not of fixed location. The upper fifth of the height is of decreased section, and a light tower caps the building. The steel frame is braced by brackets from columns to main cross-girders, and by deep lattice and plate girders in the external walls.

Building II. —A 26-story building, rather irregular in ground plan, may be considered as very roughly rectangular, 58 x 100 ft.; 325 ft. high to roof. The longest dimension in plan is E-W. The building stands at a southeast corner, and has fairly free exposure on all sides. A building adjoining to the south rises to the level of the fifth floor. A partial cross-wall adds to the stiffness in N-S direction. The following values are estimated averages:

w 205,000 lbs. per ft.; I for walls 350,000 ft.^{4}; for columns, 6,000 ft.^{4}. Taking E for walls at 2,000,000 lbs. per sq. in.,
for walls, EI—968,000 x 10^8, lbs. x ft.^{2}
for columns, EI 260,000 x 10^8, lbs. x ft.^{2}

Total, EI—1,228,000 x 10 ^{8}lbs. x ft^2.

$$\frac{w}{EI} = 0.167 \times 10^{-8} \, ft^{-2}$$

$$T = \frac{6.28 \times 325^2}{10^4 \times 1.875^2}\sqrt{\frac{0.167}{32.2}} = 1.36 \text{ sec.}$$

On the basis of E — 1,500,000 lbs. per sq. in. for the walls, T — 1.56 sec

This is considerably below the observed rule. Some inaccuracies of estimate may have been committed, but these are probably not sufficient to account for the difference. The building is somewhat irregular in the upper six stories than below, the width being decreased in one step and the length in another nearer the top. It is not easy to estimate whether these variations from regular form would increase or decrease the period; it seems likely that they would operate to increase slightly the calculated value. The rear (east) wall of the building is unbroken by windows; all the other walls contain a full complement of windows.

The steel frame has one row of portal-braced panels, and several panels of loop-rod diagonals; but the latter are probably of little effect in the direction of vibration.

Building III.—A 17-story structure of rectangular plan continuing unchanged to the top. Outside dimensions about 65½ ft. x 375 ft., height from curb to roof, 206½ ft. All four outside walls contain windows. Two median cross-walls increase the stiffness. The longest dimension is N-S. The building stands in a fairly free exposure. No buildings adjoin.

The following values are estimated averages: w — 878,000 lbs. per ft.; I for walls—2,211,000 ft.^{4}, for columns—5,642 ft.^{4}

At E=2,000,000 lbs. per sq. in. for the walls Total EI—6,612,000 x 10.^{8}; and $\frac{W}{EI}$ — .1325 x 10.$^{-8}$

At E — 1,500,000 lbs. per sq. in. for the walls. Total EI— 5,020,000 x 10^8; and $\frac{W}{EI}$ = .175 x 10^{-8}

$$T = \frac{6.28 \times 206.5^2}{10^4 \times 1.875^2}\sqrt{\frac{0.1325}{32.2}} = 0.49 \text{ sec.}$$

or

$$= \frac{6.28 \times 206.5^2}{10^4 \times 1.875^2}\sqrt{\frac{0.175}{32.2}} = 0.56 \text{ sec.}$$

These are only about one-third to one-fourth the measured value. The estimate is approximate throughout, but can involve no errors sufficient to account for more than a small part of the discrepancy.

This building is peculiar in this, that one-half is of very heavy wall-bearing construction, while the other end is of steel-frame skeleton construction, though it also has unusually heavy walls. The proportional action of each part is quite problematical. The measurements were made at about the middle of the length of the newer (steel-skeleton) portion. The bracing of the frame is a portal system of rather shallow girders. The possibility of torsional vibrations in such a composite structure may be conjectured.

After the preceding calculation was made, the writer re-examined the records from this building for a motion of shorter period. The records obtained in this building are few in number, and these made during very light winds, so that no great variety was available. At several points, however, very minute traces of a small, evanescent motion were found, apparently of period 0.45 to 0.6 sec. Unfortunately, as at best only two or three consecutive waves were to be found, accurate measurement was not possible, and therefore the criterion of constancy of period could not be applied to fix the character of these traces of motion. Moreover, their evanescence, and the persistent appearance and far greater amplitude of the longer wave (period 1.8 sec.), is in contrast with the records from the other building, if the minute motion be taken to represent the lateral vibration of the building as a whole. While the evidence thus is very weak, yet it seems more important when the shape and construction of the building, and the total disagreement of the calculated value of 0.48 to 0.59 sec. and the observed value 1.8 sec. are considered.

If the indication referred to be truly chargeable to integral lateral vibration of the structure, so that 0.45 to 0.60 is to be substituted for the 1.8 sec. previously recorded, then for this building the comparison of computed and observed periods of vibration gives as satisfactory results as for Buildings I and II.

These observations and calculations, while exceedingly fragmentary in spite of the large amount of labor they involved, tend to show the following facts: First, that tall buildings of modern type are very completely elastic as concerns their resistance to transverse forces, and that under the influence of wind and other forces they are in almost continual vibration of substantially simple-harmonic type. Second, that the period of the transverse vibration is comparable with, but rather larger than, the period calculated from the dimensions and weight of the structure considered as an elastic bar fixed at the base, the resistance of walls and columns being computed in ratio of their moduli of elasticity. Third, that, in view of the latter fact, the strength of the walls is by no means negligible in estimating the resistance of such a structure to lateral distortion.

It is thought that the recorded observations of vibration periods of three actual tall buildings may prove of some assistance in the study of the resistance of tall buildings to earthquake motions. Statistical information as to the periods and amplitudes of the lateral components of earthquake vibrations will, of course, be essential in such study, and should be collected by those who have access to it.

ILLUSTRATIONS

Fig. 1. Vibration Records from Building I.

A—April 16, 1905; sky overcast; apparent wind, W. S. W.; Weather Bureau record, N. W., hourly movement 26 mi.

B—April 17, 1905; brisk to high wind W. and S. W.; Weather Bureau record, W., hourly movement 36 mi. (max. for 5 mins., 42 mi. hr.)

C—April 30, 1905; clear; brisk wind W. and S. W.; Weather Bureau record, W., hourly movement 30 mi.

D—May 9, 1905; wind, Weather Bureau record, N. W., hourly movement 25 mi. (N-S. pen broken.)

Fig. 2. Vibration Record from Building II Feb. 7, 1905; brisk wind.

Fig. 3. Vibration Record from Building III. July 11, 1905; cloudy, light easterly wind; Weather Bureau record, S. E., hourly movement 10 mi. (N-S. pen broken).

Fig. 4. Record of Wind Pressure against a Tall Building.

Point of measurement about 230 ft. from ground, Nov. 30, 1905. Wind West, brisk.

A—Pressure at west window.

B—Vacuum at south window.

Fig. 5. Deflection Curves of Elastic Reed of Uniform Cross-Section under Different Conditions, for Equal Maximum Deflection.

1.—Free vibration, no intermediate node.

2.—Free vibration, one intermediate node.

3.—Static deflection under uniform horizontal loading.

4.—Static deflection under horizontal load at end.

A Piece of Reconstruction Work at Stanford University

By Charles B. Wing*

FROM an engineering standpoint, one of the most interesting effects of the earthquake at Stanford was the distortion without destruction of three arches forming the arcade from the Law Building to the Administration Building, between the Memorial Court and the Interior Court of the Inner Qaudrangle. A general view of these arches is shown by Fig. 1, and details of the main piers by Fig. 2.

After the earthquake, the top of the gable over these arches was found to be 7½ inches and the caps on top of the piers 4 inches to the north. A slight crack in the masonry at the left and right abutments respectively of the smaller arches and considerable spalling of the columns, piers and carved capitals were the principal evidences of damage. Mention should be made, however, of a few loose stones near the roof where the wall plates were anchored to the masonry.

A close examination of Fig. 2 will show that the arches during the shock were lifted clear of the piers, and that the columns started to fall to the south and the square portions of the pier to the north. As the arches descended, these pieces were caught and held in the position shown by the photograph.

In considering the problem of placing these arches back in their original position and replacing the broken stones of the caps, columns and piers, two methods of securing the desired result were evident, one to take down the masonry and rebuild it in its position, and the other to attempt moving the arches back as

*Reprinted from the California Journal of Technology, University of California. Vol. 8, No. 3, January, 1907.

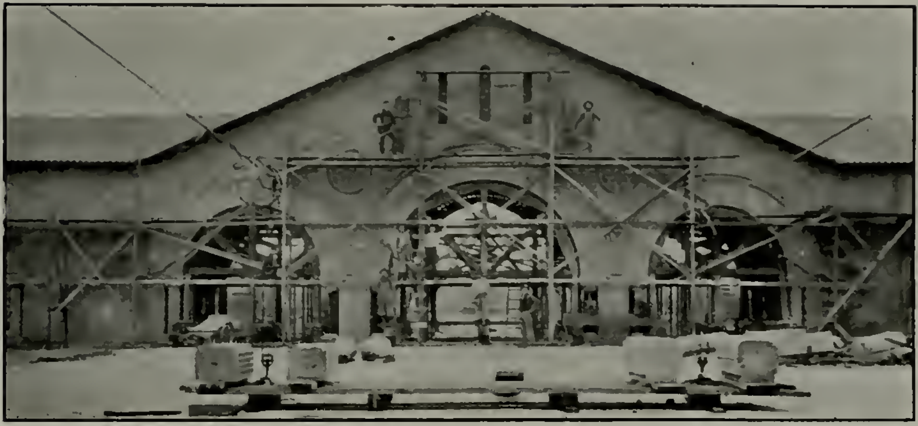


FIGURE 1. GENERAL VIEW OF ARCHES AT END OF MOVEMENT
Showing Chain Blocks Used For Pulling Down Cable to Move the Gable Walls

a whole without injuring them. As the arch rings and masonry above were practically undamaged, it was decided to try the latter method and, if injury to the masonry resulted, to rebuild the portions damaged after they were brought back into place. Centers were therefore placed under each of the arches of sufficient strength to sustain the load of from forty to fifty tons that is supported by each pier. These centers were supported at three points by brace frames of heavy timber resting on two horizontal timbers placed one above the other on the ground. These two horizontal timbers had iron plates with 1-inch steel rollers between them. The upper timber was in two pieces, connected by planks spiked to the sides, the space between being controlled by wedges acting against a piece spiked to the lower timber, thus providing a means of controlling the motion of the arches. A fair idea of the arrangement of these timbers can be gained from a study of Figs. 3 and 4. The ground underneath these timbers was covered with plank to distribute the pressure.

To provide for moving the gable masonry, heavy timbers with spreading planks were placed on the north side of each wall, as shown in Fig. 4. These timbers were connected by iron rods, with turnbuckles, extending through the walls and having an eye for attaching a cable at each end. The turnbuckles were adjusted until the walls were the right distance apart and then struts were placed between the walls. Steel cables were run from the rods to the south, as shown in Fig. 1, and attached to the base of the masonry pillars of the Memorial Church. After taking the slack out of these cables with a block and tackle, a brace frame was inserted to hold the cables up from the ground. In like manner, check cables were

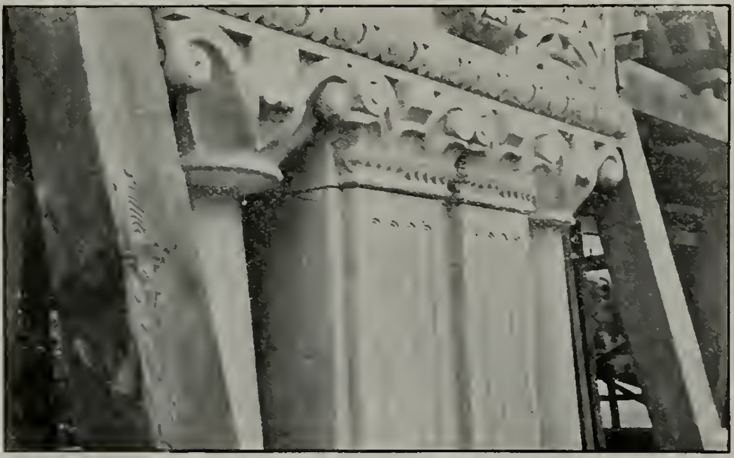


FIGURE 2. DETAIL OF LOWEST PIER OF NORTH WALL
Showing Columns Falling to the South and Square Portion of Pier Leaning to the North

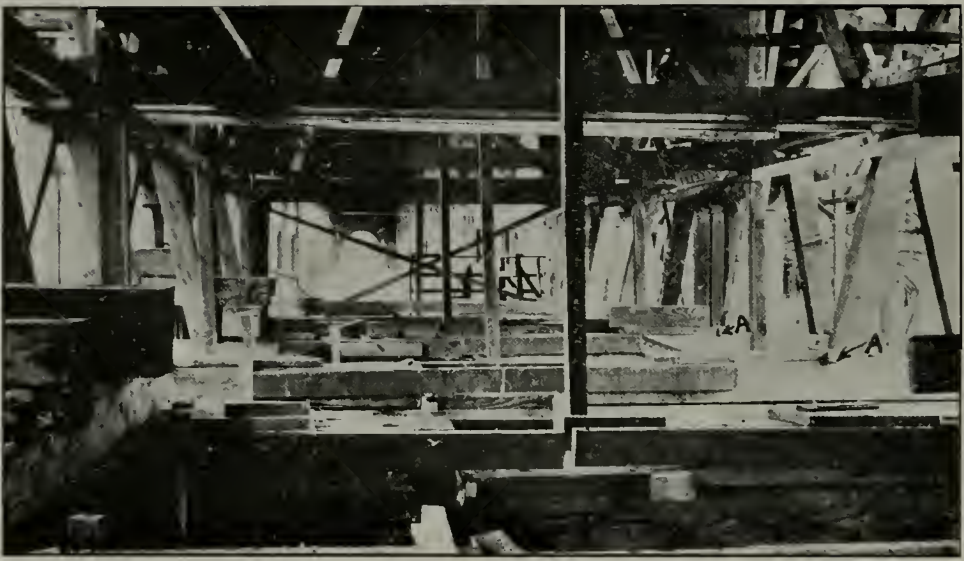


FIGURE 3. SHOWING DETAIL OF CRADLE SUPPORT ON ROLLERS
Note System of Wedges For Giving and Taking Slack as Piers Were Moved

taken to the north and the slack controlled by a brace frame, as shown to the extreme right in Fig. 4. The brace frames supporting the centers were now raised by means of wedges at points A, Fig. 3, until a greater portion of the load was carried by the centers and their supports. The mortar joints of the columns shown in Fig. 2 were then sawed out and the columns removed. The square stones of the piers were let in place as a safeguard. On moving the columns of the north wall, shown in Fig. 2, and loosening the wedges, the walls moved back on the rollers about one inch, due to the elasticity of the masonry, thus indicating the excellent condition of the masonry over the arches.

Chain blocks ware now attached to the cables leading to the church, as shown in Fig. 1. As the cables were pulled down, the gable moved over to place easily and uniformly. At the same time, by means of wedges and jacks, the cradles supporting the arches moved on the rollers until the arches were in place. The first strain was put on the cables on Thursday morning, December 13th, and on Friday night, December 14th, the arches were in place, save for a few slight adjustments. Since that time the broken stones in the caps, columns and

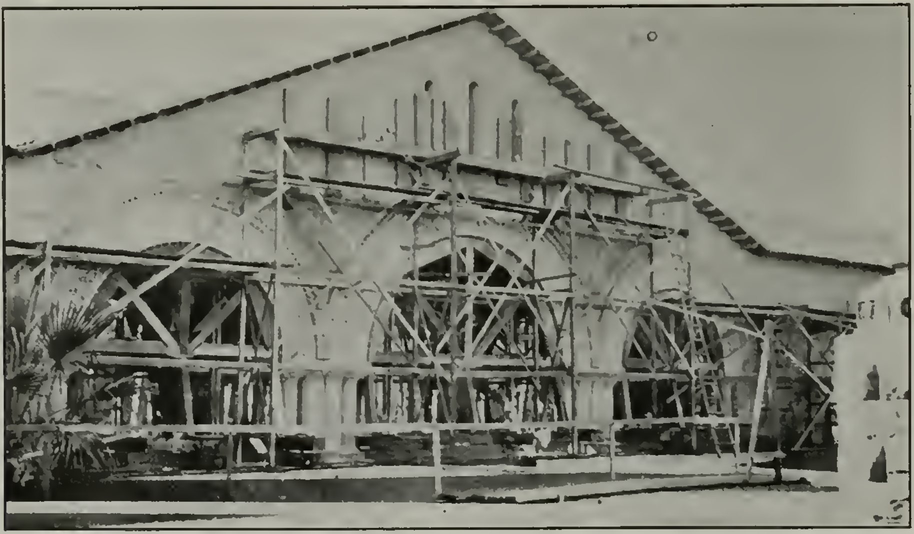


FIGURE 4. SHOWING METHOD OF SPREADING PULL ON CABLES
Over the Gable Masonry

piers have been replaced by new material. Transverse timber brace frames will be placed between the walls over the center of the main arch and over each pier in order to provide some slight additional security against distortion by the next earthquake, should there be one.

Four carpenters were employed about twelve days in placing the centers and rigging the cables, and it took seven men sixteen hours to do all the moving.

Reinforced Concrete

By W. K. Hatt *

IT was the comfortable assurance of that urbane Roman poet, Horace, that he had built himself a monument more lasting than brass in the intellectual life of mankind. At the time that he was writing these lines the Roman engineers were constructing those concrete aqueducts and domes that have served mankind on the physical side during the time that Horace had been a source of perpetual delight of the students of classical writings. Which product will endure the longer is an open question. One thing is certain, while many persons of exquisite taste may prefer Horace to our modern writers, most well-informed persons conclude that the engineer of to-day has surpassed the Roman engineer in the quality and use of concrete.

The number of recent failures of reinforced concrete buildings, attended with the loss of life of workmen, does not constitute an argument against the advance of the practice of this new art, but calls attention to the need of correct theory in design and expert supervision in construction. Steel for buildings is made under highly technical methods and a searching inspection of trained men, whereas concrete for buildings may be formed by ignorant and unskilled workmen, and may be supervised by foremen who are inexperienced in the art of proportioning and mixing the ingredients. Defective material, either of cement, sand or stone, dishonest skimping of cement and poor inspection, incorrect proportioning, and a too early removal of the wooden forms from the floors moulded in cold weather, or heavily laden with stored cement and other materials, are sufficient causes to explain these failures. An increasing number of these may be expected as time goes on and untrained men who have learned their business in other lines of construction take up the work of building reinforced concrete structures. The resulting loss of life will no doubt call attention to the necessity of regulating by proper building laws this new construction, which has spread so rapidly over the country from sea to sea. In 1902, when the first published results of experimentation appeared from the laboratory for testing materials of Purdue University, one had to go far to observe instances of reinforced concrete. Last summer in Seattle the writer saw no other type of building in process of construction. At Atlantic City in 1902, when the experiments referred to were placed before the American Society for Testing Materials, there were no instances of the use of reinforced concrete in sight. Last summer at the meeting of the society one viewed the stately and beautiful Marlborough-Blenheim Hotel entirely constructed of reinforced concrete; the replacement of the steel pier by reinforced concrete piles and girders; and the construction of a new recreation pier of this type of construction. The growth has been truly marvelous. Not only has the extent of the use in bridges and buildings increased, but the variety of its application is extraordinary. In a list of constructions in which it is successfully and economically used may be included: Retaining walls, dams, tanks, conduits, chimneys, arches culverts, foundations, floors for buildings, railroad girders, highway bridges, pipes, railway ties, piles, stairs and roofs.

At the present time the underlying mechanical principles and the constants of design of beams are fairly well determined, and we wait upon the architects to express the truth of these principles in a beautiful structure. While this type of construction associates itself with the broad and simple wall spaces and low buildings of the Spanish Mission style, with surface ornaments of tiling and Mosaic, it also lends itself to important modern civic buildings. The stateliness of beauty of the Marlborough-Blenheim Hotel at Atlantic City has been mentioned. The Ingalls Building, Cincinnati,

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*A paper read before the recent annual meeting of the National Association of Concrete Users, at Chicago.

and the New Terminal Station at Atlanta, Ga,. are other examples.

Without stopping to discuss the properties of waterproofness, fireproofness, durability, etc., or the multitude of topics of interest and importance that crowd upon one's mind in connection with reinforced concrete, attention will be simply called to the mechanical principles underlying the construction.

Concrete, like stone, is weak in tension, but strong in compression at a ratio of 1 to 10. Consequently when under flexure, as in a beam, the concrete is not used economically; for it breaks on the lower side in tension before the compressional strength is utilized. A beam may be, however, strengthened, or reinforced by the insertion of a steel rod in the lower side of the beam. These rods are usually bent up near the ends of the beam so as to also reinforce the beam against the diagonal tensional stresses that occur at the ends, due to the combination of shear and direct stress.

Before the rod can come into operation during the flexure of the beam, there must be the necessary adhesion between the concrete and the rod to transfer the stress to the rod. This adhesion or bond varies from 200 pounds to 500 pounds per square inch of the surface of a plain rod, and under favorable conditions is sufficient to develop the strength of the steel in the concrete. The adhesion is thought to be more of a mechanical action than chemical, and is due to the entrance of the fine cement into the microscopic pits on the surface of the smooth rods. Many designers use artificially deformed bars, such as corrugated bars and twisted steel bars to increase this adhesion.

Some engineers haye feared the effect of long continued vibrations on the bond of the steel or the strength of the concrete.

It is well known that concrete, because of its lack of elasticity, absorbs or deadens vibrations, and the sound caused thereby. It is not probable that vibrations reach the steel. The speaker has knowledge of many experimental attempts to loosen the bond by shocks and vibrations. So far smooth bars incased in concrete that have been subjected to shocks and long continued vibrations seem not to have lost any of their original strength of bond. Likewise the concrete on the compression side of a reinforced concrete beam that has been loaded and released from load some 2,500 times to high working stresses seems not to have been substantially weakened thereby.

In this way a beam is reinforced so that both the concrete in compression and the steel in tension may be worked to their full value. Any one who has seen a plain concrete beam broken in a testing machine, and then has witnessed a test of a reinforced concrete beam will be first of all struck by the apparently greatly increased flexibility of the reinforced concrete beam which deflects ten times as much as the plain beam before showing, in the case of a dry beam, any cracks to the naked eye, and when the load is removed the elasticity of the steel draws the beam back nearly to its original shape. It is probable, however, that this process of bending the reinforced concrete beam develops very minute flaws in the concrete which are invisible to the naked eye, unless the concrete has been soaked in water, so that it is not safe to count upon a tensile strength of the concrete in developing the total resisting strength of the beam at the usual working stresses. Small fissures are no doubt present in the concrete before the stress is applied. Designers usually compute the resisting moment of the beam as based upon the compressional stresses in the concrete and the tensional stress in the steel alone.

Engineers as a rule have found it necessary to review their knowledge of mechanics in dealing with reinforced concrete, not that there is any new principle involved, but the number of factors in the equations of flexure is greater, and an account must be taken of the relative moduli of elasticity of the two materials, steel and concrete. Furthermore, the lack of perfect elasticity of the concrete under certain conditions leads to an assumption of some other than a rectilinear relation between stress and strain.

In calculating the strength of the reinforced concrete beam, sufficiently approximate results can be obtained by omitting consideration of the tensile stresses in the concrete, and supposing a rectilinear relation between stress and strain. The moment of flexure is then most simply expressed as the total force in the steel multiplied by the distance to the centroid of the compressive stresses. This latter distance is expressed with sufficient accuracy as a fraction of the depth of the beam, this fraction having been determined by experimental measurement on the steel beams.

Care in all cases must be taken to compute the maximum compressive stress arising in the concrete under the conditions of the problem, and also the amount of diagonal tension at the ends of the beams must be computed and provided for by stirrups, or by bending up some of the rods at the ends.

The simple methods of computation recommended by Professor Talbot seem to the speaker to be conservative and clear. Professor Talbot's experiments have served the purpose of fixing the principles of action of a reinforced concrete beam. It is a matter of congratulation that the extensive facilities for testing reinforced concrete in various forms in the University of Illinois are directed by one who has shown such a firm grasp of the problem, and can direct the experiments to a solution of problems of essential importance.

The question of lumber for forms in reinforced concrete construction is one of increasing importance. The scarcity of lumber is reflected in the increasing price. This is not a temporary condition, but is justified by the economic conditions. It is a well known fact that the forest resources are rapidly becoming depleted, and that we are rapidly approaching the time when we shall feel the pinch of a timber famine. Even at the present time the lumber cost for forms in reinforced concrete construction is a considerable item. The progress of design will be largely in the direction of casting the materials in permanent forms and then erecting them, and of so designing the building and centering that these may be simple and inexpensive, and that the sequence of operations will result in the use of as little lumber as is necessary. The organization of the United States which has given a large amount of attention to the study of the supply of timber and the methods of preparing it for the market, and its value for structural purposes, is the Forest Service of the United States Department of Agriculture. Members of this association who wish information concerning our timbers upplies and their quality, will do well to apply for the publications of the Forest Service. They can be had by addressing the Forester, Forest Service U. S. Department of Agriculture.

To conclude this brief consideration of reinforced concrete, a conservative estimate would include the following principles;

1. Concrete is durable and fireproof when made of the proper aggregate.

2. The strength of combinations of steel and concrete may be calculated with a sufficiently close degree of accuracy.

3. Shapely and beautiful structures may be built of this material. It is particularly adapted for mill buildings because of the absence of vibrations which are induced in the ordinary type of mill buildings by the rapidly revolving machinery.

4. The cost of a properly designed reinforced concrete building, where wooden forms are used to advantage, is said not to exceed more than 5 or 10 per cent of the cost of mill buildings of the ordinary type with brick walls and wooden beams of the so-called slow-burning construction, provided that the concrete may be laid as at present by unskilled labor.

A Code of Ethics For Architects*

SECTION 1. No member should enter into partnership, in any form or degree, with any builder, contractor, or manufacturer.

SEC. 2. A member having any ownership in any building material, device, or invention, proposed to be used on work for which he is architect, should inform his employer of such ownership.

SEC. 3. No member should be a party to a building contract except as "owner."

SEC. 4. No member should guarantee an estimate or contract by personal bond.

SEC. 5. It is unprofessional to offer drawings or other services on approval (and without adequate pecuniary compensation).

SEC. 6. It is unprofessional to advertise in any other way than by a notice giving name, address, profession, and office hours, and special branch (if such) of practice.

SEC 7. It is unprofessional to make alterations of a building designed by another architect, within ten years of its completion, without ascertaining that the owner refuses to employ the original designer,or, in event of the property having changed hands, without due notice to the said designer.

SEC. 8. It is unprofessional to attempt to supplant an architect after definite steps have been taken towards his employment.

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*Adopted and recommended to its members by the Boston Chapter of the American Institute of Architects.

SEC. 9. It is unprofessional for a member to criticise in the public prints the professional conduct or work of another architect, except over his own name or under the authority of a professional journal.

SEC. 10. Unpaid competitive work, either private or public, is detrimental to the architectural profession. Competitions should be held only under competent professional advice. Before accepting an invitation to compete, architects should agree upon conditions acceptable to all concerned, as prior acceptance may prevent just and desirable modifications.

SEC. 11. No member should submit drawings except as an original contributor in any duly instituted competition, or to secure any work for which such a competition remains undecided.

SEC. 12. The A. I. A. "schedule of charges" represents minimum rates for full, faithful, and competent services. It is the duty of every architect to charge higher rates whenever the demand for his services will justify the increase rather than to accept work to which he can not give proper personal attention.

SEC. 13. No member shall compete in amount of commission or offer to work for less than another, in order to secure the work.

SEC. 14. It is unprofessional to enter into competition with or to consult with an architect who has been dishonorably expelled from the "Institute" or "Society."

SEC. 15. The assumption of the title of "Architect" should be held to mean that the bearer has the professional knowledge and natural ability needed for the proper invention, illustration, and supervision of all building operations which he may undertake.

SEC. 16. A member should so conduct his practice as to forward the cause of professional education and render all possible help to juniors, draftsmen, and students.

Meeting of the San Francisco Chapter of the American Institute of Architects

THE February meeting of the San Francisco Chapter of the American Institute of Architects was the occasion for the first dinner given by the Chapter since the calamity of last April. The meeting and dinner were held at Tait's Cafe, and the time was spent in discussing an enjoyable menu and in the transaction of business. The principal business of the evening was the election of officers for the coming year, as follows:

Albert Pissis, President; William Mooser, Vice-President; Sylvain Schnaittacher, Secretary; Henry A. Schulze and William Curlett, Trustees.

Messrs. Schulze and Curlett, the retiring officers, were given a vote of thanks for their services to the Chapter during their term.

An informal discussion of matters pertaining to the profession and the growth and rebuilding of the city consumed the balance of a pleasant evening.

The Chapter decided, owing to the success of the evening, that meetings be held monthly on the third Thursday of each month, and that each meeting be the occasion for a similar dinner.

Among those present were: Mr. Herman Barth, Frederick D. Boese, William Curlett, B. J. S. Cahill, Clinton Day, Lionel Deane, Chas. Dickey, H. Geilfuss, William Mooser, Matt O'Brien, D. F Oliver, Albert Pissis, James Reid, P. Righetti, Sylvain Schnaittacher, H. A. Schulze, E. J. Vogel, C. Werner, T. J. Welsh and C. A. Wright.

American Society of Civil Engineers;

February Meeting; San Francisco Association

REGULAR meetings of the San Francisco Association of members of the American Society of Civil Engineers are held at intervals of two months. The first meeting of the current year will be held at Tait's Restaurant, Van Ness Avenue and Eddy Street, on Friday evening, Feb. 15, 1907.

The program for the evening, as announced, considers the discussion of recent papers published in the Proceedings of the Parent Society at New York City. Among the papers to be discussed are the following:

1. The Fatigue of Concrete, by J. L. Van Ornum.
2. The Design of the New Croton Dam, by Edward Wegmann.
3. Liberation of Air in Siphons, by Charles Anthony, Jr.

In addition to the above, it is proposed to further discuss papers which were considered at the last meeting of the Association in December, namely:

1. The Durability of Wooden Stave Pipe, by Arthur L. Adams.
2. Hydraulic Filled Dam Construction, by James D. Schuyler.

The regular business meeting will be preceded by a dinner at 5:45 P. M.

FURTHER REFERENCE TO ROMAN STONE

IN our February issue, pp. 56–59 inclusive, we presented an article with illustrations describing the process for making Roman Stone, an artificial stone made of cement along modern lines.

The manufacture of artificial stone has become a most important industry, and we are always glad to present through our columns information relating to advanced processes.

On page 59 of our February issue, unfortunately a typographical error or omission occurred. The percentages of water were omitted in the table showing strengths of concrete. As this table presented the most important fact in the article, we reprint it in full, with corrected heading.

	8%	14%	20%
7 days.........	261	392	301
28 days.........	344	447	407
3 months	344	494	430
6 months	392	536	454
12 months	300	478	474

These results show that in using less than 14% of water the strength is fully developed at six months, after which it begins to deteriorate; while mixtures made with a greater amount of water develop their strength slowly but uniformly without deterioration.

The table brings out clearly one of the advantages of wet concrete, and the manufacturers of Roman Stone are particular upon this point in their process. We regret that the typographical omission occurred in our February issue.

HIGH CLASS BRICK WORK*

THE difficulty which the wreckers are having in razing the Palace Hotel in San Francisco does not bear out the reports which are so freely circulated concerning inferior construction in that city. Most people who have traveled know the Palace Hotel. It was seven stories high, 300 x 225 feet, and was built in 1870. The bricks were laid in a mortar composed of lime and cement with strips of Norway iron and steel cables at frequent intervals. The mortar is too hard to be scattered by the cleaning machines. The supports include 100 pillars weighing seven tons each. The walls are first dynamited and then pulled down by hoisting engines—1½-inch steel cables—equivalent to the strength of 10,000 men. Notwithstanding this, the strips of wall remain unbroken on the ground and require picks and sledges to loosen the bricks. The laborers working below the surface are frequently compelled to desist because of fire which is uncovered. In many places the combination of wood, plaster and mortar has formed a sort of charcoal, which, secluded from air by piles of debris, has been smoldering ever since the fire on April 18th. Boiling water or steam is often found in these places. Sand seems to pulse and throb at the head and the crowds of laborers are constantly on the alert to avoid being burned or scalded. There are 10,000,000 brick in the building and the debris will fill 80,000 wagons. The contractor is receiving $30,000 for wrecking the building, and the value of the salvage is estimated at $20,000.

So much has been said and written of a derogatory nature in reference to past building construction in San Francisco, that we reprint with much pleasure the above extract from the editorial columns of a recent number of Construction News of Chicago. The facts, as printed, are no doubt painted rather brightly, and the picture of boiling water and throbbing ground should perhaps be taken with a grain of salt. However, the fact remains that the brick work of the Palace Hotel was a noble and honorable piece of work, and construction interests in California should welcome more construction of this type.

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*Reprinted from the editorial columns of "Construction News," Vol. 23, No. 5, Chicago, Feb. 2, 1907.

LEGAL STATUS OF ARCHITECTS *

In a paper on the "Legal Status of Architects" in the Architect and Builders' Magazine, John E. Brady, of the New York bar, cites numerous cases to show "that an architect has no power to alter or modify the contract of construction, and that his acceptance of the work after the contract has been so changed, is not binding upon the parties." Mr. Brady also reaches the conclusion that "it is universally held that an architect superintending the erection of a building has no inherent authority to bind his employer by accepting work inferior to or different from that agreed upon in the contract, and that such authority can not be read into a clause in the contract making the architect's approval a condition precedent to the builder's right to recover." Mr. Brady cites several interesting cases, of which the following is reprinted:

"In a Connecticut case the facts showed that a builder entered into a written contract to do all the work necessary for the construction of a house according to definite plans and specifications and for a fixed sum. A provision of the contract constituted the architect, who drew the plans, superintendent of construction, and his acceptance of the work was made a condition precedent to the liability of the owner therefor. The architect ordered the builder to make certain alterations necessitating extra material and labor, and it appeared that the owner had no knowledge of the extra work until after the completion of the building. It was held that the builder could not recover as against the owner for any work or material furnished which was not called for by the contract. The contract defined the exact extent of the architect's authority. Under it he was given no power to order alterations and, in so doing, he was acting beyond the scope of his agency. The case stands for the proposition that, when a builder furnishes material or performs labor at the request of the architect, he assumes the risk of the architect's having proper authority or of obtaining the ratification of the owner."

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*Reprinted from Vol. 6, No. 5, of Rock Products, Louisville, Ky., Feb. 5, 1907.

PARTITIONS FOR OFFICE BUILDINGS*

By L. A. Norris

There are several types of partition, which I believe have proved satisfactory; among them are the solid lath and plaster partition, the double lath and plaster partition, the reinforced concrete partition; and the reinforced terra cotta partition.

Solid Plaster Partitions.—I believe that the solid plaster partition should be constructed of rods, channels or angles, spaced 12 to 18 inches on centers according to the grade of lath which is used, and that these channels should extend from the concrete slab on the floor to the bottom of the slab above, or where suspended ceilings are used should be securely fastened to the suspended ceilings and should be lathed on one side with suitable lath, and should then be plastered with a good grade of gypsum plaster or with cement mortar; that is, about 4½ parts of good lime mortar, gauged with one part Portland cement.

These metal partitions should be in all cases framed around openings and should not in any way depend upon the wooden bucks for support. And grounds that are used in such partitions should run along side studs perpendicularly to the floor rather than horizontally, their area and lengths being as small as possible.

Hollow Partitions.—Where metal hollow partitions are constructed, the rigidity of the partition depends to a large extent upon the metal studding work, and this work should be constructed in such a manner that the partition will be rigid before the plaster is applied.

Concrete Partitions.—Concrete partitions can be constructed where required throughout a building, and should not be less than two inches thick, and should be reinforced with some suitable reinforcing, which should be rigidly held in place, inside of the centering before the concrete is poured in.

I believe the most satisfactory way to construct concrete partitions is to build them in a building before the floor above is installed, allowing the reinforcing to be embedded in the upper floor.

It is hardly practicable to install a suitable concrete partition after the floors are all in, as the concrete should be poured into the centering from the top.

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